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WASTEWATER ENGINEERING AND MANAGEMENT PLAN FOR BOSTON HARBOR - --ETC(U)

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WASTEWATER ENGINEERING AND MANAGEMENT PLAN

FOR

BOSTON HARBOR - EASTERN MASSACHUSETTS METROPOLITAN AREA

EMMA STUDY

TECHNICAL DATA VOL. 7
COMBINED SEWER OVERFLOW REGULATION

OCTOBER 1975

INDEX TO EMMA STUDY REPORTS

COVER PHOTOGRAPH

The photograph on the cover of this Technical Data Volume depicts the [illegible] Park Overflow with a view of Carson Beach in the background.

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WASTEWATER ENGINEERING
AND MANAGEMENT PLAN
FOR
BOSTON HARBOR – EASTERN MASSACHUSETTS METROPOLITAN AREA
EMMA STUDY.

TECHNICAL DATA [redacted] Volume 7.
COMBINED SEWER OVERFLOW REGULATION.

FOR THE

METROPOLITAN DISTRICT COMMISSION

COMMONWEALTH OF MASSACHUSETTS

BY

METCALF & EDDY, INC.

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CHAPTER 1

INTRODUCTION

General

Boston Harbor and the rivers tributary to it have been the prime resources responsible for the growth of the Boston metropolitan area. These waters have for many years served industrial, commercial and recreational activities providing, among others, the service of wastewater disposal. This has resulted in the deterioration of these resources to the degree where competing uses have suffered.

One of the major causes of pollution recognized for many years has been overflow from combined sewers. Initially, combined sewers were built to convey sewage and stormwater, two urban nuisances, to the nearest watercourse. In 1884, the Boston Main Drainage Works were completed consisting of interceptors collecting much of this pollution and diverting it to the then newly constructed Moon Island facilities for discharge away from the shoreline in deeper waters. By about 1900 additional interceptors were constructed which diverted stream and shoreline discharges to deeper waters off Deer and Nut Islands constituting respectively the North Metropolitan Sewerage District and the South Metropolitan Sewerage District.

These interceptors were generally sized to carry all dry weather flow plus an additional allowance for stormwater. The stormwater was believed to dilute the dry weather flow to the point where overflows would not adversely affect the quality of the receiving waters.

One of the most comprehensive early studies on the conditions in the Boston Harbor and its tributary streams was reported in Massachusetts House Document No. 1600 of 1936.* At that time no treatment was provided for any discharges to the Boston Harbor.

Findings at that time among others were that bacterial pollution, floating solids, slick and sludge deposits were the factors related to objectionable conditions in the

*Report of the Special Commission on the Investigation of the Discharge of Sewage Into Boston Harbor and Its Tributaries, Massachusetts House Document No. 1600, December 1936.

Harbor, but in no case did results show that a nuisance would result from lack of oxygen.

As part of its recommendations, the report recommended provision for treatment at the main interceptor outlets, preparation of adequate works to remove causes of overflows that prevent bathing along the waterfront of Boston Harbor and its estuaries and tributaries except at such points as may meet with the approval of health agencies.

Today primary treatment is being provided to the intercepted flows at the Deer and Nut Island treatment plants. However, numerous locations exist in the Boston Harbor area where, during rain storms, combined sewage overflows into the receiving waters untreated as it did in 1936.

Recognizing the importance of this source of pollution the New England States and the Environmental Protection Agency have established the following policy and program recommendations on combined sewers and urban runoff.

Policy

"The New England States and the EPA recognize combined sewer discharges and urban runoff as a major water pollution control problem in New England. Joint State-Federal water pollution control programs should place special emphasis on the control and elimination of these discharges through construction and operation and maintenance programs, giving priority to those discharges affecting bathing and shellfish. EPA should continue funding demonstration projects. In addition, the States and EPA recognize the necessity for programs to minimize the pollutional impact of urban storm runoff."

Program Recommendations

1. "Accelerate Municipal Planning for Combined Sewer Control.

2. Accelerate Municipal Programs for Operation and Maintenance and Construction to Control or Eliminate Combined Sewer Discharges.

3. Give Appropriate Priority to Combined Sewer Correction in the State-Federal Planning Process and Construction Grants Program.

4. Clarify the Types of Treatment Required for Combined and Storm Sewer Discharges.

5. Alleviate Pollution from Urban Runoff in Designing Combined Sewer Correction Systems and by Encouraging Local Land Management Practices and Regulatory Measures.

6. Achieve Consistent Policies and Design Standards for Combined Sewer Correction Programs among State and Federal Agencies Involved in Combined Sewer Correction."*

Purpose

It is the purpose of this technical data volume to present an evaluation of the combined sewer overflow problem in the Boston Harbor area in terms of quantifying the problem and identifying the direction that technical, environmental and economic analyses should take during detailed facilities planning.

Project Approach

Due to the highly varying nature of combined sewer discharges both in terms of flow and pollution concentrations, the approach to this project is to use computer models to aid in the quantification of overflows and sizing of alternative remedial facilities.

The approach to the project is one of design by analysis wherein computer simulation is used to determine the magnitude of the combined sewer overflow problem under design conditions. Once the magnitude of the problem is determined alternative pollution abatement programs can be analyzed, cost estimates based on uniform treatment processes for all alternatives can be prepared and environmental benefits can be assessed.

*"Joint State-Federal Policy and Program Recommendations for Four Key Determinants of Water Quality in New England," Region 1, U. S. Environmental Protection Agency and New England Interstate Water Pollution Control Commission, June 1974.

The basic tool used to generate the quantity and selected quality parameters for each overflow under design conditions was the Environmental Protection Agency's Storm Water Management Model (SWMM).*

Overflows were developed through the use of the two major blocks of the SWMM. These blocks, designated as RUNOFF and TRANSPORT, are modified versions of the original model to permit more effective use. A third, EXECUTIVE block, was used to provide the function of interprogram coordination.

The RUNOFF block represents processes that occur from the time rainfall begins to the point where runoff enters the main sewer system, taking into account groundwater infiltration, surface detention and overland flow. Quality constituents of surface runoff (i.e., Biochemical Oxygen Demand (BOD), suspended solids (SS) and coliform) are also included in the model.

The TRANSPORT block represents the flow processes that occur in conduits, manholes, and various control structures of the main sewer system. The quality of surface runoff and domestic wastes flowing through the transport system is accounted for in terms of concentrations of BOD, SS and coliform bacteria counts. Effects of sediment deposition and pickup along with BOD decay are also included.

Simulation of the water quality in the Boston Harbor as a result of combined sewer overflows was done by the Massachusetts Division of Water Pollution Control. Overflow hydrographs obtained as output from the SWMM were used as input to the Division's Hydrodynamic and Time Variable Water Quality Model** to simulate conditions in the Harbor. Simulation included existing conditions as well as conditions for each combined sewer abatement alternative, and indicated the effectiveness of alternative pollution control measures.

*Storm Water Management Model, Vols. I-IV, U. S. Environmental Protection Agency 11024DOC07 to 10, prepared by Metcalf & Eddy, Inc., University of Florida, and Water Resources Engineers, Inc., July 1971.

**Development of Hydrodynamic and Time Variable Water Quality Models of Boston Harbor, Commonwealth of Massachusetts Water Resources Commission, prepared by Hydroscience, Inc., July 1973.

Report Structure

This report, Technical Data Vol. 7, Combined Sewer Overflow Regulation, is organized into six chapters and six appendixes. Chapter 2 contains descriptions of the combined sewer service area within the MDC system, the extent of the overflow problem and its effects on the Boston Harbor receiving waters. Analysis Criteria used in the computer simulation to determine the quantity and quality of overflows are presented in Chapter 3. Descriptions of existing and overflow pollution control facilities and recent studies are presented in Chapter 4. Chapter 5 presents a description of additional alternatives and related costs. Also contained in this chapter are the results of modeling the Boston Harbor water quality. Chapter 6 presents a recommended plan of action.

Appendix A presents the method of design hyetograph development. Appendix B contains a comparison of results obtained by using design storms of 1-year and 15-year frequency. Demonstration of the SWMM in the study area is presented in Appendix C. Overflow characteristics of the drainage areas as obtained from the computer simulations are presented in Appendix D. Appendix E contains computer modeling instructions for the user of the SWMM. An index of the modeling packages used to simulate the combined sewer area is presented in Appendix F.

CHAPTER 2

EXISTING SITUATION

General

The combined sewer area within the MDC system is composed of a densely populated urban area supported by one of the oldest sewer collection systems in this country. Except for small isolated sections, combined collection systems were constructed prior to 1940 with the oldest sections dating back to before 1900.

Today, combined sewers serve about 50 percent of the MDC served population covering about one-fifth of the sewered area tributary to the MDC systems.

Overflows of combined sewage occur in excess of 100 locations.

During a recent conference on the Boston Harbor*, it was presented that the biggest problem confronting the Boston Harbor area is solving the combined sewer discharge problem, which was proposed as the number one water pollution control priority.

Combined Sewer Area

The combined sewer area in Metropolitan Boston consists of all or parts of five communities. These five communities together with abbreviations used to identify them for modeling purposes are listed below:

Municipality	Abbreviation
Boston	B
Brookline	BR
Cambridge	C
Chelsea	CH
Somerville	S

Figure 2-1 shows the extent of the combined sewer service area and the breakdown of the area into separately modeled drainage basins. This breakdown was necessitated by the modeling requirements. Each basin was assigned an identification code which is also shown on Figure 2-1 and in Table 2-1.

*Proceeding, Third Session, Conference In the Matter of Pollution of the Navigable Waters of Boston Harbor and Its Tributaries - Massachusetts, Environmental Protection Agency, October 1971.

FIG. 2-1 COMBINED SEWER AREA

TABLE 2-1. COMBINED SEWER DRAINAGE BASINS IN THE BOSTON METROPOLITAN AREA

Municipality	Basin(1)	Area(2) (acres)	Sewered population (x 1,000)
Boston	B1	690	11.1
Boston	B2	1,000	8.6
Boston	B3	1,295	34.0
Boston	B4	1,234	98.7
Boston	B5	835	110.9
Boston	B8(3)	2,425	235.9
Boston	B9(4)	2,370	39.9
Boston	B10	1,085	4.6
Boston	B11	2,296	32.5
Boston	B12	2,152	65.6
Boston	B15	3,098	48.5
Boston	B16	1,423	53.7
Chelsea	CH	460	10.1
Brookline	BR	497	17.4
Somerville(5)	S1	1,286	36.9
Somerville	S2	423	14.2
Cambridge	C3	603	16.5
Cambridge	C7	100	3.5
Cambridge	C8	23	2.8
Cambridge	C9	137	4.2
Cambridge	C10	382	12.2
Cambridge	C12	220	7.2
Cambridge	C14	221	2.9
Cambridge	C16	118	2.9
Study area total		24,370	875.0

1. Basins B14, B17 and C4 shown on Figure 2-1 are separate sewer areas and are not included in this combined sewer analysis.
2. Includes separate sewer areas tributary to combined sewer systems.
3. Includes B8A and B8B.
4. Includes B9A and B9B.
5. Somerville Basin S1 includes Cambridge Basin C1.

Also shown on Figure 2-1 are the approximate locations of 69 selected overflow points that were included in the models. Although not all minor points were explicitly modeled, their flow contributions were accounted for and aggregated with the larger nearby overflows in the models.

Although overflow locations exist along the North Charles Relief Sewer upstream from the existing Cottage Farm Combined Sewer Detention and Chlorination Station in Cambridge, these were also excluded as they are expected to be taken care of at that facility. The impact of these overflows on the Charles River Basin needs to be monitored in light of the ongoing separation program in Cambridge and in relationship to other discharges in the Basin.

In Boston, all or parts of West Roxbury, Roslindale, Mattapan and Hyde Park are served by separate sewer systems. Most of Brookline and parts of Cambridge also have separate sewers. These separate sewer areas are not included in this study except if they are tributary to a combined sewer system. In those cases, dry weather flows are also included. Table 2-1 lists the drainage basins containing combined sewers together with their area and population. Drainage basin identification numbers in Table 2-1 correspond to those shown on Figure 2-1.

Dry weather flow transported by the combined sewer system is routed to the City of Boston and MDC interceptors and transported to the Deer Island Treatment Plant. During wet weather, combined sewage overflows to the following receiving waters:

1. Charles River
2. Mystic River
3. Chelsea River
4. Neponset River
5. Boston Harbor
6. Dorchester Bay.

Overflow Occurrences

Stormwater characteristics are highly variable in quantity and quality. Overflows at a location are dependent on the degree of rainfall, antecedent conditions, season of the year and, to some degree, on the capacity of the interceptor system serving it.

During a study on rates of flow at the MDC East Boston Pumping Station* it was found that stormwater runoff affected flows at the pumping station about 9 percent of the time.

Also, McKee** reported that about 6 percent of the time, overflows could be expected in the Boston Harbor area. Studying selected rainfall records for summer recreation periods, he concluded that for interceptors designed on the basis of three times dry weather flow about five overflows per month would occur.

Extent of the Overflow Problem

A general comparison of the quality of combined sewer overflows with other urban wastewaters is shown in Table 2-2.

As shown in Table 2-2, in terms of average biochemical oxygen demand for the 15 cities measured, the quality of combined sewage overflows equals that at Deer and Nut Island treatment plants discharges but is considerably greater in terms of the average rate of solids discharged.

Similar comparisons can be made for biochemical oxygen demand and suspended solids from the results of the combined sewer modeling and effluent loadings from Deer Island Treatment Plant. The following loads were estimated to be overflowing to the receiving waters during simulation of design storm conditions. A runoff period of approximately 8 hours was used since the one-year design storm produces runoff detectable by the SWMM for approximately 8 hours.

Combined Sewer Overflows

Peak flow	13,000 cfs (8,400 mgd)
Total loads	
BOD_5	75,100 lb

*"Memorandum to the Metropolitan District Commission Relative to Pumping Head and Rates of Flow at East Boston Pumping Station", Metcalf & Eddy Engineers, April 10, 1940.

**"Loss of Sanitary Sewage Through Storm Water Overflows, by J. E. McKee, Journal Boston Society of Civil Engineers, 1947.

TABLE 2-2. QUALITY COMPARISON OF COMBINED SEWAGE WITH OTHER URBAN WASTEWATERS

	BOD_5 (1) mg/L	SS (2) mg/L	Total coliforms (3) MPN/ 100 ml
Combined sewage			
Average (4)	115	410	5×10^6
First flush (4)	170-182	330-848	1.5 to 310 x 10^6
Extended flow (4)	26-53	113-174	1.5 to 310 x 10^6
Surface runoff (4)	30	630	4×10^5
Deer Island plant effluent (5)	107	68	- (6)
Nut Island plant effluent (5)	119	103	- (6)

1. Biochemical oxygen demand is the amount of oxygen resources required for reducing organic matter and, therefore, is a measure of strength of organic pollution. Concentrations shown are in milligrams per liter.
2. Suspended solids are a measure of matter discharged and consists of both biologically degradable and inert particles. Normally, the proportion of inert particles is greater in combined sewer overflows than in treatment plant discharges. Concentrations shown are in milligrams per liter.
3. Total coliforms are harmless bacteria used as indicators of the probability for disease-producing organisms to be present. A measure of their presence is used as criteria to prohibit swimming and shellfishing. Concentrations are reported as most probable number per 100 milliliters.
4. Based on measurements in 15 similar urban areas in this country and reprinted in Urban Stormwater Management and Technology: An Assessment, U. S. EPA, EPA-670/2-74-040, prepared by Metcalf & Eddy, Inc., December 1974.
5. July 1, 1974 to June 30, 1975.
6. Negligible due to disinfection of effluents.

SS	795,480 lb
Coliform	10^3 MPN/100 ml to 10^8 MPN/100 ml

These loads, as simulated by the SWMM, can be compared to the recorded Deer Island Treatment Plant effluent loadings for 1975 to better understand their impact. The SWMM has been demonstrated in cities similar to Boston such as Chicago, Philadelphia, San Francisco, Washington, D. C. and Cincinnati. Further, actual measurements from the Lowell Street drainage area in Cambridge were used to demonstrate the reliability of the SWMM calculated results (see Appendix C).

Deer Island Treatment Plant

Maximum hourly rate	990 cfs (640 mgd)
Average daily flow	450 cfs (292 mgd)
Total loads	
BOD_5	86,600 lb/8 hr (260,000 lb/day)
SS	54,600 lb/8 hr (164,000 lb/day)
Coliform	99.99 percent kill

More important than biochemical oxygen demand and suspended solids is the discharge of floating matter and the failure to provide disinfection of combined sewer overflows.

Although pollution from combined sewer overflows is only intermittent and has been estimated to occur on the average only about five or six times per month for short periods of time, the location of such discharges, as shown on Figure 2-1, makes them a deterrent to the effective use of the Boston Harbor as a recreation resource. The problem is not the overall volume of pollution discharged annually, which is relatively small compared to the volume of the receiving water in the Harbor, but the intermittent discharge of floating matter, undisinfected fecal wastes, debris and other solids constituting a danger to health and aesthetics of the Boston Harbor water resources.

A measure of the aesthetics problems is the numerous complaints that Boston Harbor is polluted by the population in general.

A measure of the health aspects of combined sewage overflows is reflected by coliform measurement data at the time of and shortly after rainstorms. Such data, however, are limited. An analysis of recent data relative to Boston Harbor and the Charles River Basin is presented in the following sections.

Effect of the Overflows on the Boston Harbor. Coliform bacteria data collected by the Division of Water Pollution Control* during the summer and fall of 1972 in the Boston Harbor indicates sensitivity of total and fecal coliform to rainfall. Data from six representative gaging stations were reviewed and the results indicate a generally consistant pattern with coliform counts increasing after heavy rainfall.

A typical inner harbor station located 200 yards off the Aquarium and Central Wharf recorded total coliform counts ranging from 240,000 to 930,000 most probable number per 100 milliliters (MPN/100 ml) on three separate days when 0.5 inches of rain or more had occurred within the previous 36 hours. At the same station on three other days with rainfall of less than 0.1 inch occurring in the previous 36 hours total coliform counts ranged from 24,000 to 93,000 MPN/100 ml. This data suggests that the increase in rainfall resulted in combined sewer overflows causing an increase in total coliform count by a factor of 10.

A typical station in Dorchester Bay just west of Thompson Island recorded a similar response of a ten fold increase in total coliform count after moderate to heavy rainfall. On three separate days with rain of at least 0.5 inches in the previous 36 hours total coliform counts ranged from 93,000 to 24,000 MPN/100 ml. On four days with 0.1 inch of rain or less in the preceeding 36 hours the total coliform count ranged from 91 to 2,400 MPN/100 ml. Results for fecal coliform counts were found to follow the same pattern as total coliform for the 1972 study.

*Boston Harbor Pollution Survey - 1972 Part A: Data Record of Water Quality and Wastewater Discharges, Division of Water Pollution Control, Massachusetts Water Resources Commission, April 1973.

Additional data was collected during the summer of 1970 by the Massachusetts Department of Public Health and is presented in a report on the Dorchester Bay Beaches.* This data indicate that rainfall as well as tide conditions affect the total and fecal coliform counts in the near vicinity of the Dorchester Bay Beaches. In the case of Tenean Beach (the only beach where bacteriological quality of water was found to be unsatisfactory as a result of the Public Health Study), it was recommended that bathing be restricted during periods of low tide water or after rainfalls of certain minimum intensities.

Effects of the Overflows on the Charles River. The Charles River Basin, comprising approximately the lower 8 miles of the River, was intended to become "the most beautiful and useful park in America." Serious deterrents to this goal have been the pollution entering the Basin from upstream discharges, combined sewer overflows to the Basin and admission of polluted salt water at the Dam.

Planning is underway for the upgrading and constructing of treatment plants in the upstream areas of the Charles River.

The new Charles River Dam being constructed just downstream from the present Dam is designed to reduce the salt water intrusion and thereby eliminate stratification of the basin by drawing discharge water from submerged intakes.

The combined sewer overflow problem has begun to be abated by the operation of the Cottage Farm Combined Sewer Detention and Chlorination Station, located just upstream of the Boston University Bridge in Cambridge. The Cottage Farm Facility treats overflows from the North and South Charles relief sewers. Much still needs to be done to abate combined sewer overflows to the Basin, particularly, as will be seen, in the Back Bay Fens which forms an important part of the Charles River Basin.

*Report to the Interagency Task Force on the Survey of the Dorchester Bay Beaches, Massachusetts Department of Public Health, 1970.

Data presented in the report on Combined Sewer Overflows to the Charles River Basin* indicates that the coliform count in the Basin increases during and immediately following rainfall. The largest increases were recorded within and just downstream of the Back Bay Fens indicating the importance of controlling combined sewer overflows entering the Fens. Controlling overflows into the Fens means for the most part controlling overflows from the Stony Brook conduit which drains more than one third of the Charles River Basin. Overflows from the Stony Brook conduit are discharged into the Fens approximately 3/4 of a mile from the Charles River.

*Combined Sewer Overflows to the Charles River Basin, Massachusetts Division of Water Pollution Control, prepared by Process Research, Inc., August 1972.

CHAPTER 3

ANALYSIS CRITERIA

General

This chapter presents the general engineering criteria used in estimating the quantity and quality of flows and the treatment processes necessary for adequate pollution abatement for combined sewer overflows from the Boston Metropolitan Area.

Design Storm

Wet weather flow in combined sewer systems consists of storm runoff and normal dry weather flow. Studies* have indicated that hydraulically, storm runoff is by far the most significant component; therefore, the selection of the hydrologic basis for design is important. In the design of storm drains and combined sewers extreme events are normally selected as the basis for sizing of pipes. Events of between five and 50 year severity have been used for such designs. For the regulation of combined sewer overflows, severities of such magnitude normally are not justified because the older combined systems normally do not have such transport capability; because the cost of regulating such flows increases drastically with severity, whereas relative pollution loads only increase slightly; and because during such hydrologically severe events the receiving water usually acts quite differently than under normal dry weather flow conditions.

For the purposes of sizing facilities and estimating their cost in this study, a storm of one-year severity and six-hour duration, as shown on Figure 3-1, was chosen as the design storm. A description of its development is presented in Appendix A. This storm was selected judgementally, based on past experience and on a comparison between a one- and a 15-year storm. Figure A-2 shows the 15-year severity storm, while Appendix B presents a comparison of the one-year and 15-year storms.

During Step I facilities planning in each area the design storm parameters should be tested for their sensitivity to cost effective pollution control and achievement of water quality objectives, including the functioning

*Urban Stormwater Management and Technology: An Assessment, U. S. Environmental Protection Agency EPA-670/2-74-040, prepared by Metcalf & Eddy, Inc., December 1974.

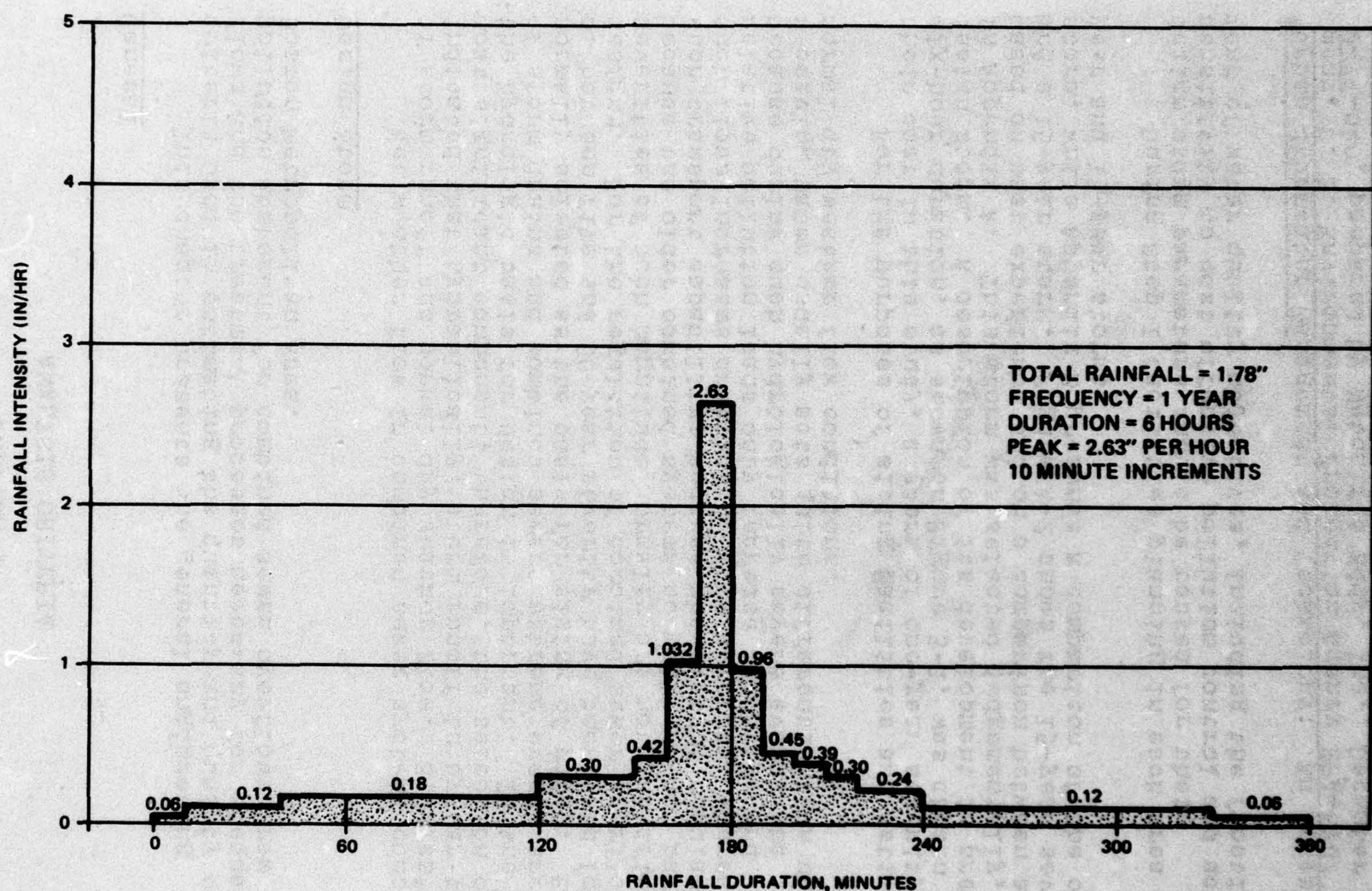

FIG. 3-1 1-YEAR 6-HOUR DESIGN HYETOGRAPH

of finally selected facilities under actual longer range hydrologic records. For example, it may be that the criteria for overflows to the Charles River Basin, the Dorchester Bay beaches and the immediate Inner Harbor should differ in accordance with their intended uses.

Dry Weather Flow Loads

The dry weather flow (DWF) loads are based primarily on the estimated area population and flows which are developed and presented in Technical Data Vols. 1 and 2. In a combined sewer system DWF is hydraulically insignificant compared to the storm flow which typically exceeds the DWF by 50 to 100 times.

Pipes sized to carry these significantly larger storm flows, permit a considerable amount of solids to settle out during dry weather low flows. In addition, opportunities exist for large debris to enter and be deposited in such systems. During storm flows, such deposits then get flushed into the receiving water through overflows. These effects are shown in Table 2-2.

In estimating the pollution loads under design storm conditions a seven day dry weather flow period allowing deposition has been assumed prior to initiation of the storm. The seven day dry weather period was judged to be a conservative estimate of time between storms. McKee* estimated that overflows of mixed untreated sewage and stormwater would occur five to six times a month on the average.

The important measures of pollution contributed from DWF in this case are the pollution loadings associated with BOD_5, SS and coliforms. Average values used in the model are as follows:

BOD_5	0.20 pounds per capita per day
SS	0.22 pounds per capita per day
Coliform	6.2×10^6 most probable number per 100 milliliters (MPN/100 ml)

Daily and hourly factors were applied to the average DWF values as listed in Table 3-1 in order to account for their variations.

*McKee, Loss of Sanitary Sewage through Storm Water Overflows, Journal BSCE, April 1947.

TABLE 3-1. DAILY AND HOURLY CORRECTION FACTORS FOR SEWAGE DATA[1]

	Day	Flow	BOD	SS	Coliform
1	Sunday	0.960	1.000	1.000	1.000
2	Monday	1.080	1.000	1.000	1.000
3	Tuesday	1.050	1.000	1.000	1.000
4	Wednesday	0.900	1.000	1.000	1.000
5	Thursday	1.040	1.000	1.000	1.000
6	Friday	1.000	1.000	1.000	1.000
7	Saturday	0.970	1.000	1.000	1.000
	Hour				
1		0.740	0.850	1.050	1.100
2		0.670	0.710	1.050	0.640
3		0.630	0.600	1.100	0.450
4		0.590	0.410	0.500	0.870
5		0.540	0.460	0.660	0.540
6		0.560	0.490	1.330	0.480
7		0.670	0.720	1.100	1.290
8		0.960	0.870	0.880	1.180
9		1.420	0.770	1.030	1.370
10		1.190	1.570	0.910	1.490
11		1.200	1.020	0.660	1.300
12	Noon	1.150	0.870	0.630	1.120
13		1.170	0.910	0.940	0.890
14		1.110	0.940	0.940	0.580
15		1.080	1.070	1.050	0.450
16		1.150	1.070	1.050	0.670
17		1.210	1.140	1.160	0.960
18		1.230	0.990	0.940	1.180
19		1.250	1.450	1.330	0.840
20		1.210	1.160	1.220	1.010
21		1.170	1.550	1.440	2.820
22		1.150	1.290	1.100	1.770
23		0.880	0.990	0.880	0.840
24		1.070	1.600	1.050	0.710

1. **Storm Water Management Model, EPA 11024DOC07, Vol. III, prepared by Metcalf & Eddy, University of Florida, and Water Resources Engineers, Inc., July 1971.**

Surface Runoff Quality

Surface runoff, once considered as clean water, contains a relatively large composition of contaminants which cannot be ignored in water quality management (see Table 2-2). According to recent studies*, the magnitude of the three pollution components of surface runoff considered in this report are characterized as follows:

1. BOD_5 content of runoff equals about the strength of domestic sewage after secondary treatment from the same land use.

2. SS content of runoff is generally about three times that of untreated sewage, but consisting mostly of inorganic materials.

3. Coliform content of runoff is about one fourth to one half the magnitude of untreated sewage. However, it is two to five orders of magnitude higher than is considered safe for water contact recreation.

In the modeling of the combined sewer systems, allowances were made for pollution from urban runoff based on general data for land use categories recommended in the EPA Storm Water Management Model and on street cleaning practices in Boston.

Percent Impervious

The percent impervious varies from a low of 20 percent in areas including parks to as high as 80 percent in the downtown areas. These values are based on:

1. Inspection of available maps and aerial photographs.

2. Values used in recent engineering reports for the area.

3. Knowledge of the area.

Other Criteria Used in the Storm Water Management Model

The following standards were followed when modeling the Boston Harbor combined sewer areas:

*Studies summarized in Urban Storm Water Management and Technology: An Assessment, U. S. EPA, EPA-670/2-74-040, prepared by Metcalf & Eddy, Inc., December 1974.

1. Subcatchment size: In each drainage area subcatchments were divided based on the natural overland flow patterns and access to the sewer system. When possible subcatchment sizes are kept less than 30 acres.

2. Pipe size: All pipes 24 inches and larger were modeled. In some cases smaller size pipes were modeled in order to more accurately define a subcatchment.

3. Surcharge limit: A general review of the sewer drawings indicated an average depth of cover to be about 10 feet*. All sewers were, therefore, allowed to surcharge a maximum of 10 feet at each manhole before surface ponding was assumed to occur. All areas were spot checked to confirm the validity of these surcharge values and in some locations, such as Somerville, specifically selected surcharge limits were used on the basis of case by case evaluations before allowing surface ponding.

4. Hydraulic parameters: The following criteria were used in the Storm Water Management Model:

   Surface retention storage

   a. impervious areas: 1/16 inches (in.)

   b. pervious areas: 1/4 in.

   Small pipes storage

   An allowance of 1/4 in. of storage over the tributary area was made to account for storage in pipes not modeled.

   Ground infiltration, as used in Horton's Equation**

   a. maximum rate: 3.0 inches per hour (in./hr)

*Drawings from the Metropolitan District Commission as well as the Cities of Boston, Somerville and Cambridge were reviewed.

**Storm Water Management Model, EPA 11024DOC07, Vol. I, prepared by Metcalf & Eddy, University of Florida and Water Resources Engineers, July 1971.

b. minimum rate: 0.52 in./hr

c. exponential decay rate: 0.00115 per second ($sec.^{-1}$)

Flow friction factors (n) as used in Manning's Equation

a. overland flow

(1) impervious surfaces: 0.013

(2) pervious surfaces: 0.25

b. existing sewers

(1) brick: 0.016

(2) concrete: 0.016

Model Applicability

During the development of the Storm Water Management Model (SWMM) various urbanized areas similar to this area were modeled for verification of the SWMM against measured data.

In order to demonstrate the SWMM for its application to the Boston Harbor combined sewer areas a selected area where limited measurements exist was modeled under measured conditions. The findings supported use of the model in this project and are presented in Appendix C.

Computer modeling results under design storm conditions are presented in Appendix D and instructions for computer modeling are presented in Appendix E.

Overflow Regulation Objectives

At the present time there exists no clearly defined criteria nationally on combined sewer overflow regulation.

However, it is expected that required correction of overflow problems will be defined in terms of meeting water quality standards. Therefore, the requirements for regulation of combined sewer overflows would vary from location to location depending on the nature of the receiving water and its intended use, but would not be subject to the "minimum secondary" criteria.

Determination of treatment requirements for each specific location is considerably more complex in the case of combined sewer overflows than for normal dry weather flow pollution due to the intermittent, storm-dependent, nature of the discharge. Not only is the discharge highly variable in terms of flow, but also in terms of pollution concentrations. For example, the initial overflow normally contains a higher degree of pollution caused by the flushing of deposits in the sewers.

Also, the effects on the receiving stream are more complex in a combined sewer overflow regime due to both hydrologic and water quality slug loads. No longer can impacts on water quality be assessed purely on an average steady state basis, but must consider time varying effects. On the other hand, present water quality criteria are related to steady state conditions and do allow for the probability of not meeting criteria occasionally. How these criteria relate to the stormwater problem has yet to be decided.

For this study, the major objectives for regulation of combined sewer overflows are:

1. Elimination of disease producing organisms,

2. removal of floating matter, and

3. reduction of solids.

It is expected that removal of heavy metals and toxics will be accomplished through regulation of industrial discharges by source control.

Basis of Cost Estimates

Estimated costs of facilities are based on January 1975 prices relating to an Engineering News - Record Construction Cost Index (ENR) value of 2200. An allowance of 25 percent for engineering and contingencies has been included in the estimated total cost.

Costs developed for each major facility considered were based on recent construction and operation of such facility and are not based on detailed estimates of each component.

Cost bases for major components follow:

Conduits. Costs per linear foot were based on cost tables in Technical Data Vol. 2. In some cases, an allowance for pile foundations was added to the conduit cost to arrive at the total cost shown.

Tunnels. Cost per linear foot of tunnel was based on unit costs for labor, equipment and materials depending upon the construction method employed and the soil or rock conditions.

Tanks. The cost per cubic foot of usable tank volume was primarily based on review of the Cottage Farm Combined Sewer Detention and Chlorination Station itemized construction cost data and adjusted to ENR 2200. In comparison with costs of such facilities, this was found high, but retained as the cost in this case. Equipment for screening, chlorination and cleaning was included in this procedure.

Pumping Stations. The cost of pumping stations was based on the pumping station cost curve presented in Technical Data Vol. 2. For stations of capacity greater than those shown $10,000 per million gallons per day was used at ENR 2200.

Separation. The cost of sewer separation of $35,580 per acre was based on the estimated regional cost of separation for the New England region as reported in a study on urban stormwater management*.

Operation and Maintenance. Operation and maintenance costs for all proposed facilities were based on material and cost data in the FY 1975 MDC budget adjusted to ENR 2200.

*Urban Stormwater Management and Technology: An Assessment, U. S. Environmental Protection Agency EPA-670/2-74-040, prepared by Metcalf & Eddy, Inc., December 1974.

CHAPTER 4

REMEDIAL OPTIONS

General

This chapter reviews past options on combined sewer cverflow regulation and expands on additional remedial opportunities.

Early Studies

As presented in Chapter 1, Massachusetts House Document No. 1600 of 1936 evaluated pollution problems in Boston Harbor and its tributary streams. This study inventoried in detail pollution problems and sewerage systems in the Boston Harbor area identifying 217 external overflow locations, 165 internal overflow locations to relief conduits, and 154 regulators. Major factors relating to pollution conditions were identified as:

Bacterial pollution,

floating solids,

slick, and

sludge deposits.

Dissolved oxygen content in the waters was not found to be a nuisance anywhere. However, the oxygen content in the Fort Point Channel was found to be lowest of the readings taken.

As a result of this study, a comprehensive engineering study* on sewerage and sewage disposal also involving combined sewer overflows in the entire Boston Harbor metropolitan area was conducted. This was reported on as part of Massachusetts House Document No. 2465 of 1939. Following these engineering studies, preliminary designs** were prepared for selected projects.

*Report to The Special Commission Established by the Legislature in 1938 Upon Sewerage and Sewage Disposal in Metropolitan Boston, Greeley & Hansen and Metcalf & Eddy, Engineers, March 1939 (Published as part of House Document No. 2465 of 1939).

**The Commonwealth of Massachusetts Metropolitan District Commission, Sewerage and Sewage Disposal Studies Under Chapter 512, 1939, General Design Drawings and Final Report, Metcalf & Eddy and Greeley & Hansen, January 1941.

Relative to combined sewer overflows, remedial alternatives were investigated. Treatment of overflows were considered as impractical. Separation of combined sewers were found too expensive. The recommended plan was to construct storm overflow conduits. The most critical problems resulting from combined sewer overflows were found to be in the Charles and Mystic river basins and in the Fort Point Channel area of the Harbor.

Along with storm overflow conduits, the report recommended fine screening and chlorination at selected overflow discharge points, namely, at the East Boston Electric Pumping Station, at Back Bay Fens gate houses and at the Charles River Dam. In addition, it was recommended that provision be made for circulation of Charles River Basin water into the Back Bay Fens area and the Broad and Lechmere canals.

Two of these projects have been recently restudied and modified to accommodate existing conditions. For example, the recommended fine screening and chlorination of combined sewer overflows at the Charles River Dam have been updated to include storage and detention in the Charles River Chlorination-Detention-Pumping Station and is being constructed in conjunction with the New Charles River Dam.

Abatement of stormwater pollution in the Back Bay Fens has also been restudied for the Division of Water Pollution Control resulting in similar recommendations. Recommendations to remove accumulated sludge and provisions for aerating the Fens are underway. Long term solutions for recirculating Charles River flows and diverting Muddy River flows through the Fens and regulating overflows at the Back Bay Fens Gate House are under consideration.

The screening and chlorination at the East Boston Electric Pumping Station to abate stormwater pollution and circulation of Charles River water into Broad and Lechmere canals have not been implemented.

Existing Facilities

Facilities presently available to provide additional capacity during wet weather flows are:

1. The East Boston pumping stations and the North Metropolitan Trunk Sewer providing about 120 million gallons per day (mgd) capacity to divert flows from upstream areas to Chelsea Creek or to Deer Island (See Technical Data Vol. 9).

2. The City of Boston Calf Pasture Pumping Station and Moon Island discharge facilities providing about 155 mgd* design capacity to divert wet weather flows to the Boston Harbor before overflows occur in the immediate shoreline areas.

3. The Cottage Farm Combined Sewer and Chlorination Station designed to treat a maximum rate of 233 mgd prior to overflowing into the Charles River Basin.

4. The Somerville Pretreatment Facilities designed to treat about 160 mgd prior to overflowing to the Mystic River tidal waters.

The first two are actually old facilities retained as standby facilities intended for emergency use. The remaining two have recently been put into operation for purposes of combined sewer overflow pollution abatement and are described further as follows. In addition, a combined sewer overflow treatment facility is presently under construction on the Cambridge side of the Charles River, and, for the purposes of this study, is considered as existing and is described under this section.

Cottage Farm Combined Sewer Detention and Chlorination Station. This station was placed in operation in 1971 and is typical of the design for the Charles River Marginal Conduit Chlorination-Detention-Pumping Station under construction in Cambridge. The Cottage Farm facility is designed to treat flows up to 233 mgd from the North and South Charles Relief Sewers and the Charles River Valley Sewer. Flows in excess of the Relief Sewer capacities discharge at existing overflow locations. Overflows to the station are pumped into the facility and receive treatment by screening, skimming, chlorination and sedimentation before being discharged to the Charles River. Overflows retained in the tanks after each storm are returned to the interceptor system for transport to Deer Island Treatment Plant. An advantage of the Cottage Farm facility is that even during dry weather flows if there is a malfunction at the main pumping facilities flow can be diverted through the chlorination-detention tanks before overflowing into the Charles River Basin.

*The Special Commission Established by the Legislature in 1938 upon Sewerage and Sewage Disposal in Metropolitan Boston, prepared by Greeley and Hansen and Metcalf & Eddy, 1939.

Somerville Pretreatment Facilities. Consisting of the Somerville Marginal Conduit and Chlorination Station, this station is designed to treat overflows to the Mystic River from Somerville and is equipped with screening and chlorination facilities. The chlorine contact time is achieved in the conduit which diverts these flows to the Mystic River below the location of the Amelia Earhart Dam.

Charles River Chlorination-Detention-Pumping Station. The Charles River Project, presently under construction near the new Charles River Dam, is designed to abate pollution from combined sewer overflows into the Charles River Basin and from the Cambridge and Boston marginal conduits, the Millers River Overflow from Cambridge and Charlestown and the Lowell Street Overflow from Boston. Flows up to 323 mgd will receive treatment consisting of screening, chlorination, settling and skimming for purposes of removing solids and destructing bacteria. Discharges will be to the Boston Inner Harbor below the new Charles River Dam, also under construction.

Combined Sewer Separation Projects

Certain areas served by combined sewers have been or are undergoing sewer separation.

Boston has separated sewers in its urban renewal areas. As part of urban renewal Charlestown is undergoing separation.

The Town of Brookline has carried out separation in certain areas.

The City of Cambridge reportedly is undergoing separation in parts of the City not tributary to the combined sewer regulation facilities there.

Recent Studies

Considerable attention to combined sewer overflow abatement has been given both nationally and locally. Recent studies in the Boston Harbor metropolitan area on combined sewer overflows regulation are summarized as follows.

The Deep Tunnel Plan*. This concept, developed for the City of Boston, proposes to collect all the combined sewer overflows, both in Boston and the surrounding cities and towns of Cambridge, Somerville, Chelsea and Brookline, and transport them via transmission tunnels into a 17.3 mile system of deep rock tunnels. The system, sized to handle flows up to a 15-year storm, would intercept overflows via a system of transmission tunnels and a storage tunnel and route all flow to a pumping station at Deer Island where the flow would be screened, chlorinated, and pumped to an outfall conduit. The outfall conduit would discharge through diffusers to the ocean approximately 9 miles seaward from Deer Island and beyond the Graves.

The Old Harbor Area Pollution Control Plan**. This proposal developed for the City of Boston is designed to reduce overflows to the Carson, M Street, L Street and City Point beaches by diverting combined sewer flows through a series of gravity collection conduits and force mains to the Reserved Channel. Prior to discharge the overflow would pass through a chlorination-detention facility.

The Tenean and Malibu Beach Water Quality Improvement Plan***. This proposal for the Massachusetts Division of Water Pollution Control is for a marginal conduit which would divert overflows away from the Tenean and Malibu beach areas to the Columbus Park area where it would be connected to either the proposed Old Harbor Pollution Control Project facilities or the proposed Deep Tunnel System and would depend upon the completion of either of these projects.

*Report on Improvements to the Boston Main Drainage System for the City of Boston, by Camp, Dresser & McKee, September 1967.

**Water Quality Improvement of Tenean and Malibu Beaches, Appendix H, "Proposed Harbor Pollution Control Program for the Old Harbor Area", letter report to City of Boston, by Camp, Dresser & McKee, May 1972.

***Water Quality Improvement of Tenean and Malibu Beaches, Commonwealth of Massachusetts, Division of Water Pollution Control, by Camp, Dresser & McKee, Inc., November 1972.

Water Quality Improvement Plan for the Boston Back Bay Fens*. A two phase program for improving the water quality in the Back Bay Fens was developed for the Massachusetts Division of Water Pollution Control in 1973. The initial phase, which began in November 1973, recommended rehabilitation of existing facilities to reduce overflows to Back Bay Fens, dredging of sludge from the Fens, provision for aerating the Fens, and diversion of the main flow from Muddy River through the Fens to improve circulation.

The future program recommendations were made contingent on the implementation and effect of current plans and phase one projects in the area. Phase two recommendations include: Provision of facilities for pumping overflows from the MDC Fens Gate House to the Cottage Farm Combined Sewer Detention and Chlorination Station, additional improvements for flow routing, additional dredging of the Fens, provision of facilities for chlorination in the Stony Brook Conduit, provision of an overflow detention and chlorination station at Boston Gate House No. 1, and provision of facilities for circulation of Charles River Basin water through Back Bay Fens.

Pollution Control Alternatives for Dorchester Bay**. A series of actions have been recommended to the Metropolitan District Commission for pollution control in the Dorchester Bay. These include: removal of sediment from the Dorchester Interceptor, completion of a series of remedial repairs and modifications to specified tide gates and regulators, implementation of a flushing system to prevent accumulation of solids in pipes determined to have a moderate to heavy tendency to have deposition, and diversion of a portion of the present overflows to another location. These actions would be taken in those parts of the sewer system belonging to the City of Boston. It was also recommended that water quality monitoring of the Dorchester Bay be implemented to insure an updated data base upon which revisions and/or additions to the original program could be based. Finally, along with the above recommendations, the continued operation of the Calf Pasture Pumping Station and the Moon Island Holding facility should be evaluated.

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*Report on the Water Quality Improvement of the Boston Back Bay Fens, for the Commonwealth of Massachusetts Division of Water Pollution Control, by C. E. Maguire, Inc., May 1973.

**A Study of Pollution Control Alternatives for Dorchester Bay, Commonwealth of Massachusetts Metropolitan District Commission, by Process Research, Inc., December 1974.

Since the recommendations are made on the premise of achieving water quality under average conditions rather than during wet weather periods when combined sewer overflows occur, the study is not specifically a plan for combined sewer overflow regulation.

Overflow Regulation Opportunities

Regulation of overflows can be through reduction in flow and/or pollution loads. Both of these goals can be achieved at the source, in the transport system, at the discharge point, or by any combination of these.

Source controls generally would constitute industrial pretreatment for the exclusion of toxic wastes, good housekeeping such as neighborhood sanitation via effective street and catchbasin cleaning, erosion control and control in the use of de-icing compounds and pesticides. Flow attenuation through land use management and provision of runoff retention on roofs, pavements and in open areas, although not generally applicable in densely developed urban area, are also means of source control.

Transport system controls include sewer separation, in-sewer storage through maximum use of facilities, control of in-system deposited materials through improved regulator designs and operation, and through sewer flushing.

Discharge controls are diversion of outfalls, storage and overflow treatment.

As mentioned in Chapter 3, it is expected that industrial pretreatment will be required to remove heavy metals and toxics.

Good housekeeping measures should be carried out in any event and are not considered as alternatives. In the combined sewer area of Metropolitan Boston, other source controls such as land use management or runoff retention are not considered feasible.

Regulation alternatives considered applicable here are sewer separation, storage, treatment, outfall diversion or a combination of these.

Sewer Separation. In the past sewer separation was considered as the ultimate answer to overflow problems. However, more recently it has been considered as a poor

alternative in many cases. Not only is it expensive, it could cost up to $1.7 billion* to completely separate sewers in this metropolitan area, but it is physically not feasible in many cases such as downtown Boston. In addition, as shown in Table 2-2, the pollution content of stormwater itself makes the quality improvements achieved by separation not comparable to the investment necessary to accomplish separation.

To overcome some of the physical constraints and reduce costs, partial separation is carried out in selected cases. This involves separating the street runoff from the combined sewer system but leaving the roof runoff, cellar drainage and other building sources to drain to the combined system. Although this is considerably less expensive, about $525 million* for the Metropolitan Boston area and the runoff component is separated from the combined system, roof runoff is left in thereby failing to provide marked improvements to water quality.

Therefore, separation must be evaluated from a water quality point of view prior to its arbitrary selection. There are, however, cases where separation is the best solution, such as remote nondowntown areas where the physical inconvenience is not great, where housekeeping measures are likely to be carried out effectively and where consolidation of overflows is costly.

Retention or Storage. Complete storage of runoff is not feasible in the Metropolitan Boston area due to the large volumes of water involved. However, at the minimum, some amount of storage is required in any event to provide flow equalization for other remedial actions. In those cases involving treatment, flow equalization is necessary due to the extreme variability of combined sewer overflows and the inability for treatment processes to react to such variable loads.

Treatment. At the present time, the technology associated with treatment of stormwater and combined sewer overflows is in a dynamic state and selection of solution strategies must account for this factor. Recognizing both the importance of combined sewer overflow regulation and

*Cost figures developed By Metcalf & Eddy and adjusted to reflect estimated costs for January 1975 at ENR 2200.

the technological problems associated with developing solutions, over $90 million has been awarded over the recent years for research, development and demonstration of related technologies.*

Selection of the extent of treatment is very important because, due to large flows from stormwater, costs can become prohibitive considering the relatively few times such treatment would be required. A nationwide assessment** of costs related to treating combined sewage overflows is shown as follows to demonstrate this.

Treatment option	Capital cost*	Operation and maintenance cost**
Disinfection	4.13	46.3
Primary treatment	54	298
Secondary treatment	68	323

*In billions of June 1973 dollars.
**In millions of June 1973 dollars per year.

Diversion. Diversion of combined sewage overflows may in some cases be appropriate due to the nature of the receiving waters. Such diversions must, however, comply with nondegradation requirements and, therefore, are not expected to be remedial options by themselves.

Selected Regulation Process

The regulation processes finally selected may differ at each location on the basis of local system nature and capacity, space availability, water quality conditions and designated water uses. Figure 4-1 shows for the combined sewer areas addressed by this study a breakdown of different areas of concern.

Regulation processes in areas tributary to Dorchester Bay and Constitution Beach must address specific health

*Urban Stormwater Management and Technology: An Assessment, EPA-670/2-74-040, U. S. Environmental Protection Agency by Metcalf & Eddy, Inc., December 1974.
**Draft Report to National Commission on Water Quality on Assessment of Technologies and Costs for Publicly Owned Treatment Works Under Public Law 92-500, Addendum, by Metcalf & Eddy, Inc., April 1975.

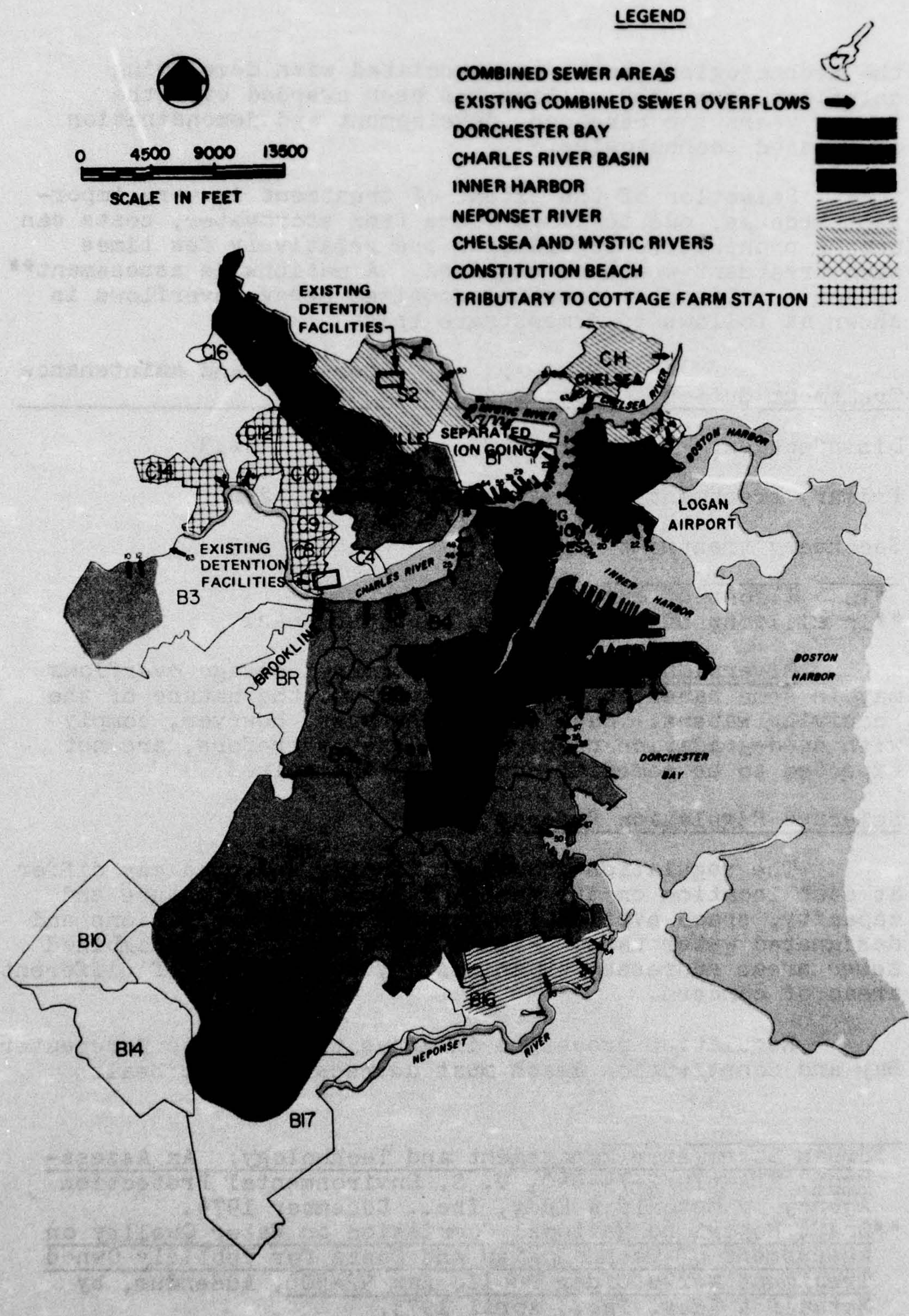

FIG. 4-1 COMBINED SEWER AREA SEPARATED BY RECEIVING WATERS

aspects and aesthetics in order to insure that constraints to their use as beach areas are removed.

In handling of overflows in the Chelsea, Mystic and Neponset River Estuary areas problems of residence time must be evaluated as well as health impacts on potential shellfish areas along with aesthetic impacts.

The Boston Inner Harbor area is primarily a commercial shipping and recreational boating area with aesthetics as the prime impact. However, movement of overflowing wastes from these areas with a potential of affecting beaches and shellfish areas must be evaluated and such effects must be constrained.

Combined sewer overflows into the Charles River Basin, including the Back Bay Fens, are yet another problem. The Charles River Basin must be considered as a storage reservoir where residence time is long in comparison to the other receiving waters. Such an effect must be included in the remedial actions for that area.

Inflows into the Back Bay Fens are primarily from urban runoff and diversion of this runoff to other locations may reduce flows in the Fens to the point of causing detrimental effects. It is recommended that the Fens serve in the future as a research and testing ground for new treatment facilities which could be used as a means of adding treated flows to the Fens to improve circulation and water quality.

For the purposes of this study, a uniform treatment process has been selected to provide a basis for cost estimates and for identification of resulting pollution discharges. The process addresses the primary pollutants of concern stated in Chapter 3 under <u>Overflow Regulation Objectives</u> and is as follows:

1. Chlorination with 15 minutes detention under design storm conditions (also providing for removal of solids and other pollutants through capture or sedimentation).

2. Screening for removal of large solids.

3. Skimming for removal of floatables.

CHAPTER 5

IDENTIFICATION OF ALTERNATIVES

General

As discussed in Chapter 4, in addition to sewer separation and recommendations on housekeeping type improvements, there have been recently two general approaches to combined sewer overflow regulation in the Boston Harbor area.

1. Consolidation of overflows at partial treatment facilities to limit pollution in immediate need areas.

2. Large scale collection and diversion of overflows for deep ocean discharge, thereby eliminating pollution from the combined sewer overflows.

The Cottage Farm Combined Sewer Detention and Chlorination Station is an example of the first approach, and the Deep Tunnel Plan, as proposed to the City of Boston, represents the second approach.

The first approach is a decentralized handling of combined sewer overflows which permits staged implementation in accordance with criteria and needs of each immediate area and provides flexibility for inclusion of future technologies in treatment beyond that presently provided.

On the basis of 38 months of operation of the Cottage Farm Detention and Chlorination Station, MDC reports average pollution constituent removals shown in Table 5-1.

TABLE 5-1. COTTAGE FARM DETENTION AND CHLORINATION STATION PERFORMANCE(1)

Parameter	Overall removal (incl. contained flows), (percent)	Removal from flow which discharged to the Charles River, (percent)
Biochemical oxygen demand	42	24
Suspended solids	45	22
Volatile suspended solids	58	37
Settleable solids	69	58
Total coliform organisms	99	99

1. Source: Cottage Farm Combined Sewer Detention and Chlorination Station, Cambridge, Massachusetts for U. S. Environmental Agency, by Commonwealth of Massachusetts Metropolitan District Commission 1975, (Draft).

Since such facilities are located in highly competitive land use areas, their design should incorporate multiple use of the land. Examples of such multiple uses exist including parking garages, recreation areas and bus stops.

The second approach is the other extreme of complete centralization and requires an early commitment to the entire plan. It, however, is intended to remove all stormwater and pollution that reaches sewers from discharging into the immediate Boston Harbor waters and diversion to Massachusetts Bay following screening and chlorination. It must be noted, however, that although the findings in the report* are that prevailing winds may be expected to carry materials that float to the surface out to sea, it would be extremely important that floating matter be removed prior to discharge. As a result of studies** for possible extension of outfalls to deep waters beyond the Graves as opposed to providing treatment, floats and chips were released from near the Graves in order to observe the possible travel of floating wastes discharged at that location.

Under certain conditions floats reached Deer Island and Green Island. Chips released near the Graves were retrieved at Winthrop, Houghs Neck, Hull, Lynn, and islands within the Boston Harbor and as far as Swampscott and Cape Cod. Findings at that time were that unless sewage discharged at that location were treated for the removal of grease and other floating matters, such matters probably would reach the neighboring shores.

Various alternatives between the previously two stated approaches were investigated and are presented in the following sections.

Those overflows tributary to the Charles River Chlorination-Detention-Pumping Station (overflows 40, 44, 25, 46, 48, 15, 52, 36 and 66) are considered regulated in the following presentation of alternatives.

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*Report on Improvements to the Boston Main Drainage System for the City of Boston, by Camp, Dresser & McKee, September 1967.

**Report of the Special Commission on the Investigation of the Discharge of Sewage Into Boston Harbor and Its Tributaries, Massachusetts House Document No. 1600, December 1936.

Alternative 1: Decentralized Overflow Regulation

Alternative 1 proposes to consolidate 44 combined sewer outfalls into 10 groups. Each group of outfalls would be connected by conduits which would transport overflows to a regulation facility for treatment and discharge to the receiving waters. Figure 5-1 shows the schematic site locations for the proposed facilities. All of these would be subjected to detailed investigations prior to final design.

Each of the regulation systems would have the following major components:

1. Collection conduits,

2. tank,

3. pumping facilities, and

4. outfall.

Collection Conduits. The collection conduits would be sized to divert design flows to the tanks. The locations shown in Figure 5-1 are only schematic and would be subject to a detailed investigation in coordination with the site selection. The method of construction to be used would also be determined during final engineering and design. For purposes of preliminary cost estimates, pipes and conduits were sized to carry peak overflows generated by the Storm Water Management Model. In cases where large pipes were needed through densely developed areas costs were developed for tunneling as well as for open cut construction and the lower of the two costs was used.

Tanks. The tanks would receive overflow from the collection conduits. Each tank would be divided into two bays for flexibility of operation. Flow would be delivered first to one bay in each of the tanks. As this bay is filled, a floating scum and oil baffle would rise with the liquid surface to maximize capture of such materials. Depending on the magnitude of a storm event, flow would then pass into a transfer channel for distribution to the second bay, or additional flow could be bypassed to the second bay permitting retention of the first flush in bay one. When the entire basin is filled, overflows would pass to the receiving stream. Using the design storm flow data, the basin cross-section and configuration would be designed to optimize velocity and settling conditions within the available volume of storage. Screens would be installed between an effluent scum baffle and the overflow weirs to polish the overflow before discharge to the river.

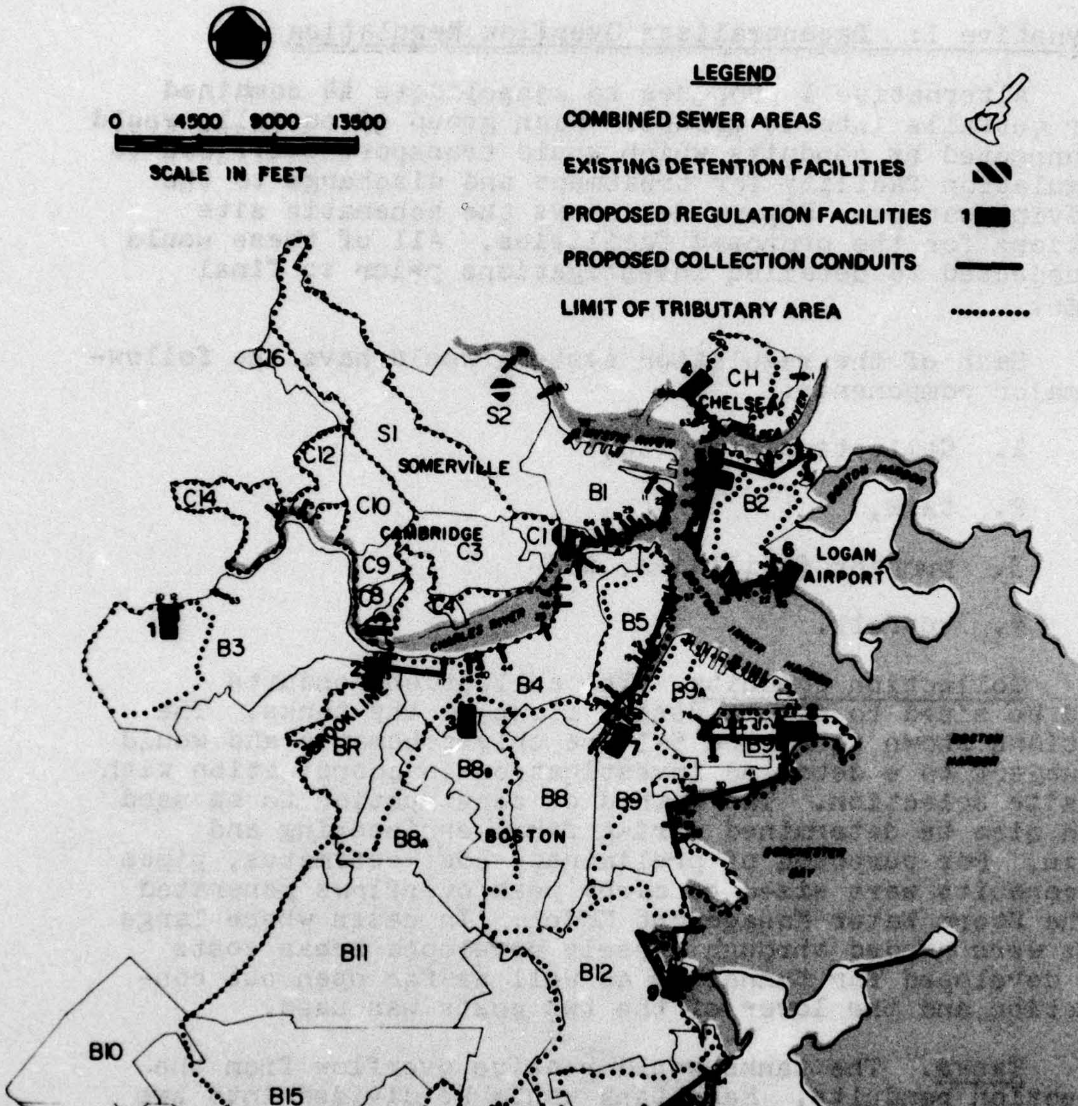

FIG. 5-1 SATELLITE REGULATION FACILITIES AND COLLECTION SYSTEMS IN ALTERNATIVE 1

The flow would be disinfected by the introduction of chlorine upstream from the tanks. The tanks would be designed to provide 15 minutes detention for the peak design flow.

Pump Station. Each regulation facility would have pumps capable of pumping the peak design flow. The location of the pumps and their capacities (before or after the tanks) would be subject to detailed investigation.

The overflow captured in the detention tanks and the solids and floatables retained would be dewatered into the interceptor system for eventual treatment at Deer Island Treatment Plant. Dewatering would be accomplished either by gravity or by pumping. Either system would be controlled by the available capacity in the receiving interceptor.

Outfall. The outfalls from the 10 regulation facilities as proposed have been located in the vicinity of the tanks. However, a detailed analysis of the receiving waters would determine if alternative outfall locations should be selected.

Table 5-1 lists the 10 proposed regulation facilities and individual outfalls.

Following is a brief description of each of the regulation facilities and their location under Alternative 1.

No. 1 - Brighton

Facility No. 1 would be located along the Charles River between the North Beacon Street Bridge and the Arsenal Street Bridge and would treat overflows 10 and 12.

No. 2 - Brookline

Facility No. 2 would be located along the Charles River in the vicinity of the B.U. Bridge and treat overflows 13 and 17. There is little available land in this area and, therefore, the facility may have to be located on the Cambridge side of the River. Outfall 13 drains parts of Brookline which are under consideration to be separated. Therefore, this development will have to be considered in the final design.

No. 3 - Back Bay Fens

Facility No. 3 would be located in Back Bay Fens at the outfall of Stony Brook Conduit at Boston Gate Houses No. 1 and 2. In order to obtain a 15 minute contact time for disinfection of the large peak flows the chlorination facilities would be placed within the Stony Brook Conduit upstream of the detention tanks. This approach is proposed for the Back Bay Fens in a study outlined in Chapter 4.*

No. 4 - Chelsea

Facility No. 4 would be located near Outfall 31 and treat Outfalls 31, 63 and 21. The discharge would be located in Island End River with subsequent discharge to the Mystic River. An evaluation of the flow in Island End River may be necessary to insure that there would be sufficient flushing action to transport the overflow to the Mystic River. Otherwise, it would be located on the Mystic River.

No. 5 and No. 6 - East Boston

There would be two facilities located in East Boston. One (north) located at the confluence of the Chelsea and Mystic Rivers treating overflows 34, 32, 3, 2, 4, 6 and 8. The second facility (south) would be located at the edge of Logan International Airport and treat overflows 18, 20, 22, 24 and 26. Treated effluent from both facilities would discharge to the Inner Harbor.

No. 7 - Fort Point Channel

Facility No. 7 would be located in the vicinity of the outfall of Dorchester Brook Conduit and treat the overflows 57, 45, 43, 19, 9 and 27 from the East Side Interceptor and Dorchester Brook overflow 33. Final design of this facility must consider the effects of planned urban renewal projects namely, Government Center, Waterfront, South Cove and South End on the combined sewer system.

*Report on the Water Quality Improvement of the Boston Back Bay Fens, for the Commonwealth of Massachusetts Division of Water Pollution Control, by C. E. Maguire, Inc., May 1973.

No. 8 - South Boston

Facility No. 8 would be located in the vicinity of Outfall 37 and discharge into the Reserved Channel. This facility would treat overflows 28, 35, 97, 38, 30, 41 and 14 presently discharging to Old Harbor and the overflows 76, 58, 42 and 37 to the Reserved Channel. Detailed investigation is needed to determine if overflows 76, 58 and 42 need to be pumped to the tank.

No. 9 - Dorchester Beaches

Facility No. 9 would be located in the vicinity of Outfall 49 and treat overflows 67, 50 and 49. The discharge location should be outside the Malibu Beach enclosure and subject to a detailed receiving water quality analysis.

No. 10 - Granite Avenue

Facility No. 10 would be located in the vicinity of Outfall 51 and treat overflows from Outfalls 54, 51 and 53. Discharge would be to the Neponset River.

For combined sewer overflows not tributary to a proposed regulation system, special localized solutions may be required. In some cases, these are already planned to be implemented and, although they may not comply with the design criteria selected for this study, are assumed to be part of the plan.

Table 5-2 shows separation as a solution for several special projects. In these cases, site specific detailed planning should be done considering other solutions. However, if separation is selected, design of drainage systems should incorporate criteria related to quality control, such as outfall structures designed to skim floatables, retain debris and large solids.

Table 5-2 applies to all alternatives considered.

TABLE 5-2. OVERFLOW ABATEMENT ALTERNATIVES: SPECIAL PROJECTS

Outfall No.	Location	Abatement alternative or existing condition
63	Brighton	Connect overflow to South Charles Relief Sewer for diversion to Cottage Farm facility.
5, 7	Cambridge	To be connected to the Cottage Farm facility via the North Charles Relief Sewer upon its completion.
93,11,23, 29,39,47, 55,64	Charlestown	These outfalls are to be separated under urban renewal projects directed by the Boston Redevelopment Authority.
88, 90	Somerville	These outfalls are treated by the Somerville Chlorination facility.
1	Chelsea	The tributary area to this outfall could be separated, possibly under an urban renewal project, or the overflow can be diverted to chlorination-detention Tank No. 6.
119	East Boston	The tributary area to this outfall could be separated or the overflow could be diverted to Tank No. 7.
95	South Boston	There is no overflow during a one-year storm at this location.

Costs. Table 5-3 summarizes pertinent data on this plan along with the construction costs of major components.

TABLE 5-3. SUMMARY OF FACILITIES AND COSTS FOR ALTERNATIVE 1

Facility No.	Location	Outfalls to be collected	Tributary area,[3] (acres)	Tank size, (million cu ft)	Construction cost (million dollars)[1] Tanks	Pumping stations	Conduits	Total[2]
1	Brighton	10,12	875	0.75	13.5	10.0	0.7	24.2
2	Brookline	13,17	980	0.38	8.5	7.6	3.6	19.7
3	Back Bay Fens	16	7,550	0.75	15.3	18.3	2.5[2]	36.1
4	Chelsea	21,63,31	330	0.18	4.4	3.3	2.9	10.6
5	East Boston (North)	34,32,3,2, 4,6,8	400	0.19	4.5	4.1	8.0	16.6
6	East Boston (South)	18,20,22, 24,26	525	0.24	5.6	4.8	3.9	14.3
7	Fort Point Channel	57,45,43, 19,9,27, 33	2,960	1.00	15.0	20.0	9.8	44.8
8	South Boston	76,58,42,37, 14,41,30,38, 97,35,28	1,075	0.68	12.8	13.6	16.9	43.3
9	Malibu Beach	67,50,49	1,580	0.68	12.8	13.6	7.8	34.2
10	Granite Avenue	54,51,53	720	0.40	9.0	9.0	4.3	22.3
	Subtotal							266.1
	Remote area projects							13.3
	Total							279.4

1. January 1975 prices (ENR 2200).
2. Includes alteration to Stony Brook Conduit and insystem chlorination.
3. Includes separate areas that discharge into combined sewers.

Alternative 2: Moon Island Tunnel Plan

In Alternative 2 regulation facilities No. 1 through 6 and 10 would exist as described in Alternative 1. Regulation facilities No. 7, 8 and 9 would be replaced by a tunnel plan shown in Figure 5-2 consisting of:

1. Surface transmission lines,
2. deep hard rock tunnels and junction chamber,
3. pump station and return sludge system,
4. chlorination-detention facilities, and
5. outfall.

A network of transmission lines would collect the overflows from combined sewers tributary to the:

1. East Side Interceptor.
2. Dorchester Brook Conduit.
3. South Boston Interceptor (North and South), and
4. Dorchester Interceptor above Victory Road.

The transmission lines would transport overflows via headworks to one of three tunnels. These tunnels would meet at a junction chamber below Columbus Circle where a fourth tunnel would transport the flow to a pump station at Moon Island. The flow would then be pumped into tanks at Moon Island. Retained flows and wastes would be returned to the interceptor system for treatment at the Deer Island Treatment Plant.

Facilities applicable to the tunnel system are described as follows.

East Side transmission conduit. This would act as a storm overflow conduit for the East Side Interceptor and discharge through a headworks into the tunnel in the vicinity of the Dorchester Brook outfall. Sizes and alignment would be similar to the collection conduit for facility No. 7 in Alternative 1.

South Boston transmission conduit. This line would collect overflows 76, 58, and 42 and transport them to the vicinity of outfall 37 where they would enter the South Boston tunnel. Sizes and alignment would be similar to the collection conduit for facility No. 8 in Alternative 1.

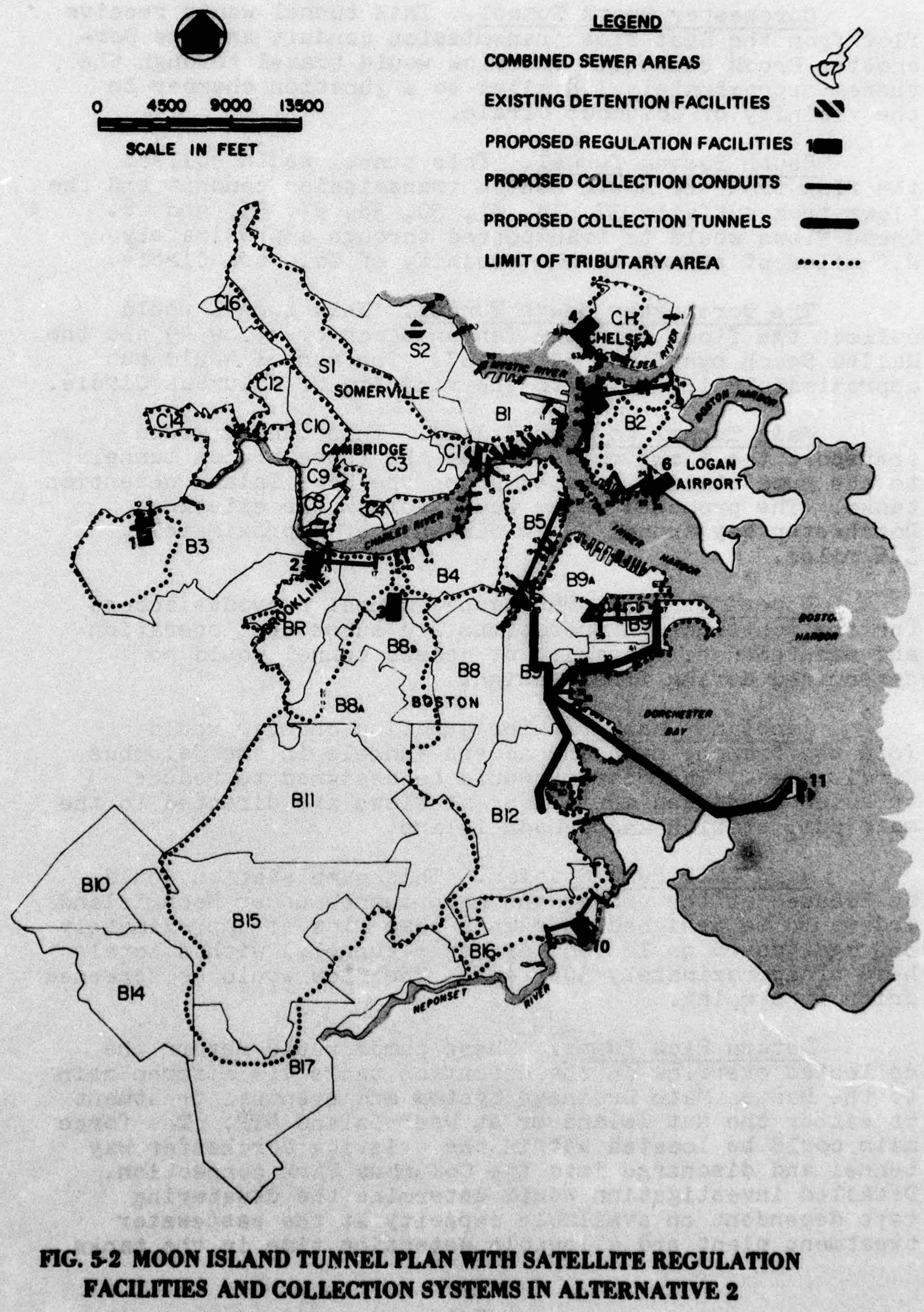

FIG. 5-2 MOON ISLAND TUNNEL PLAN WITH SATELLITE REGULATION FACILITIES AND COLLECTION SYSTEMS IN ALTERNATIVE 2

Dorchester Brook Tunnel. This tunnel would receive flow from the East Side transmission conduit and the Dorchester Brook conduit. The flow would travel through the tunnel approximately 1.8 miles to a junction chamber in the vicinity of Columbus Circle.

South Boston Tunnel. This tunnel would collect the flow from the South Boston transmission conduit and the flows from outfalls 37, 14, 41, 30, 38, 97, 35, and 28. These flows would be transported through approximately 2.0 miles of tunnel to the vicinity of Columbus Circle.

The Dorchester Beach Tunnel. This tunnel would collect the flows from the Tenean Beach overflow 49 and the Malibu Beach overflows 50 and 67. The tunnel would run approximately 1.7 miles to the vicinity of Columbus Circle.

Main Tunnel to Moon Island. This tunnel would transport the flows collected by the three branch tunnels to the pump station located under the Moon Island detention tanks. The proposed route would follow the existing Dorchester Bay tunnel for a distance of approximately 3.5 miles.

Access Tunnel. The access tunnel provides access for the construction operations and subsequent operation and maintenance. Location of access tunnel would be determined during final design.

Junction Chamber. The junction chamber would join the branch, main and access tunnels in the Columbus Circle area. The chamber would be designed to reduce turbulence and sedimentation, as flows are directed to the main pump station under Moon Island.

High Lift Pump Station. This pump station would be located at the end of the main tunnel under Moon Island and would be designed to pump a peak flow of approximately 350 mgd (based on 12 foot diameter tunnels) with a total head of approximately 300 feet. The flow would be screened prior to pumping.

Return Flow Pumps. These pumps would return the collected overflow in the detention tanks via a force main to the Boston Main Drainage System and eventual treatment at either the Nut Island or at Deer Island STP. The force main could be located within the existing Dorchester Bay tunnel and discharge into the Columbus Park connection. Detailed investigation would determine the dewatering rate dependent on available capacity at the wastewater treatment plant and allowable detention time in the tanks

at Moon Island. It is estimated that dewatering of the entire stored volume in 48 hours would require a 48-inch force main and a pump capacity of under 100 cubic feet per second (cfs).

If under final planning it is determined that only retained solids would be returned to the system dewatering of the sludge (less than 1 percent of the total volume) would require an 8 to 10-inch line with a pump capacity of at about 2 mgd.

Other alternatives to dewatering that should be investigated are on the basis of dewatering of the system at Columbus Park through back sloping of the tunnels.

Tanks. The existing tanks on Moon Island would be rehabilitated for detention storage and sedimentation.

Outfall. At the present time, Moon Island discharge is through outlet to the surface of the Harbor waters. An outfall with bottom discharge and diffusion would be provided. The exact location of the outfall from Moon Island would be subject to a detailed water quality analysis. The choice of location would seek to maximize the overflow during both flood and ebb tides.

Table 5-4 summarizes the costs, area served, and location for Alternative 2.

Costs. The major components and costs for Alternative 2 are shown in Table 5-4.

Alternative 3: Modified Moon Island Plan

Alternative 3 is a variation in the handling of Dorchester Bay overflows as shown on Figure 5-3. Tank No. 8 and 9 are combined into a single system centered around a tank at Columbus Point and upgrading of the existing Moon Island facilities. Facilities included are:

1. Stormwater collection conduits in South Boston and Dorchester,
2. tanks at Columbus Point and on Moon Island,
3. upgraded Calf Pasture Pumping Station,
4. return sludge system, and
5. outfall.

TABLE 5-4. SUMMARY OF FACILITIES AND COSTS FOR ALTERNATIVE 2

Facility No.	Location	Outfalls to be collected	Tributary area, acres(4)	Tank size, (million cu ft)	Cost (million dollars)(1)			
					Tanks	Pump station	Conduits	Total
1	Brighton	10, 12	875	0.75	13.5	10.0	0.7	24.2
2	Brookline	13, 17	980	0.38	8.5	7.6	3.6	19.7
3	Back Bay Fens	16	7,550	0.75	15.3	18.3	2.5(1)	36.1
4	Chelsea	21,63,31	330	0.18	4.4	3.3	2.9	10.6
5	East Boston (North)	34,32,3,2, 4,6,8	400	0.19	4.5	4.1	8.0	16.6
6	East Boston (South)	18,20,22, 24,26	525	0.24	5.6	4.8	3.9	14.3
10	Granite Avenue	54,51,53	720	0.40	9.0	9.0	4.3	22.3
11	Moon Island System	57,45,43,19,9, 27,33,76,58, 42,37,14,41, 30,38,97,35, 28,67,50,49	5,615		2.0(5)	42.5(5)	91.1(3)	141.6
	Subtotal							285.4
	Remote area projects							13.3
	Total							298.7

1. January 1975 prices (ENR 2200).
2. Upgrading of Moon Island plants.
3. Includes transmission lines and deep hard rock tunnels.
4. Includes separate areas that discharge into combined sewers.
5. Includes rehabilitation of Moon Island tanks including disinfectant sludge handling facilities.

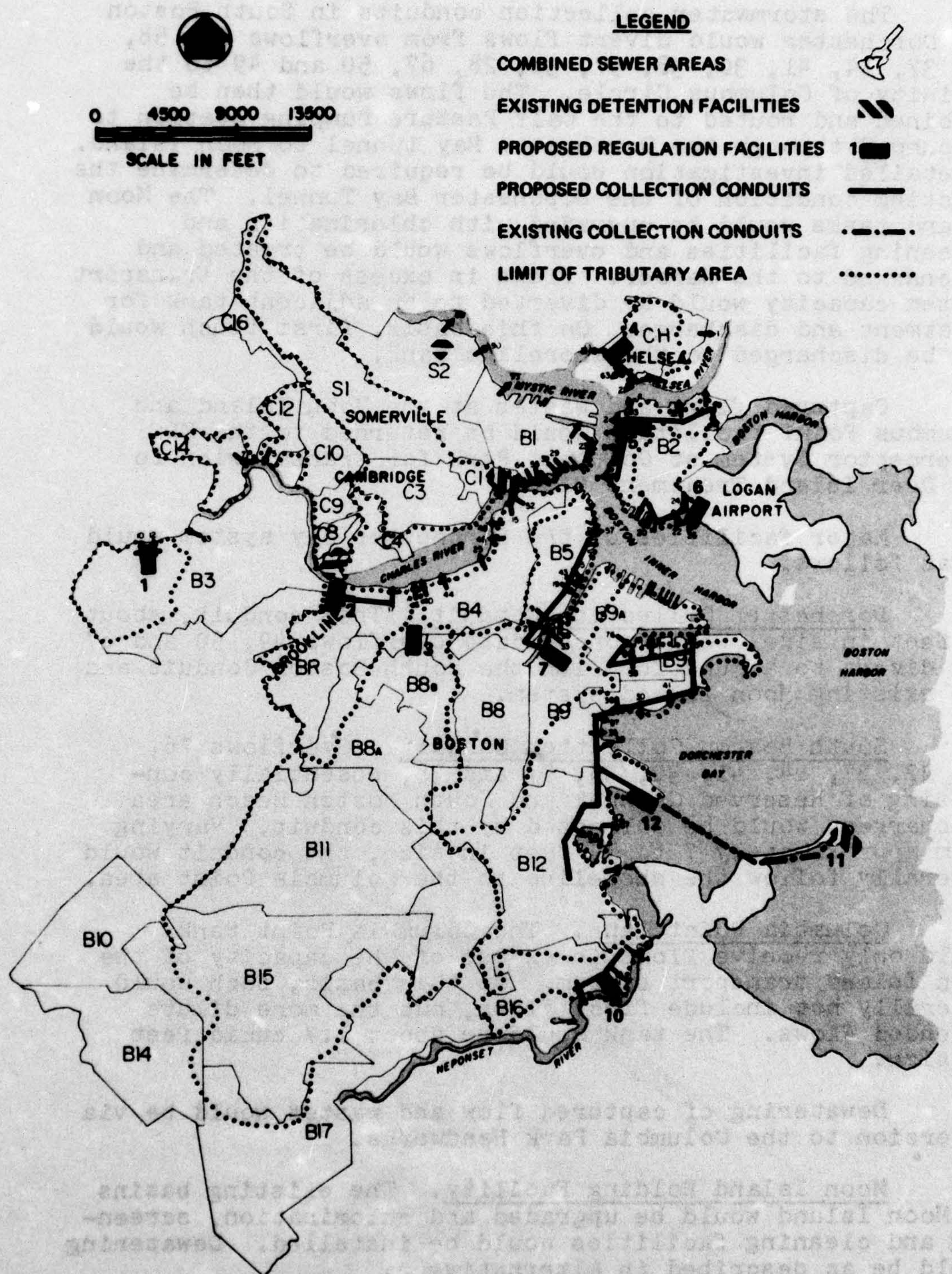

FIG. 5-3 MODIFIED MOON ISLAND PLAN WITH SATELLITE REGULATION FACILITIES AND COLLECTION SYSTEMS IN ALTERNATIVE 3

The stormwater collection conduits in South Boston and Dorchester would divert flows from overflows 76, 58, 42, 37, 14, 41, 30, 38, 97, 35, 28, 67, 50 and 49 to the vicinity of Columbus Circle. The flows would then be combined and routed to the Calf Pasture Pumping Station to be pumped through the Dorchester Bay tunnel to Moon Island. A detailed investigation would be required to determine the existing condition of the Dorchester Bay Tunnel. The Moon Island tanks would be upgraded with chlorination and screening facilities and overflows would be treated and discharged to the Harbor. Flows in excess of the transport system capacity would be diverted to an adjacent tank for treatment and discharge. On this basis, first flush would not be discharged to the shoreline tank.

Captured flows and wastes at the Moon Island and Columbus Point facilities would be returned to the MDC Interceptor System at Columbus Park for transmission to the Deer Island Treatment Plant.

Major facilities of the Dorchester Bay system would be as follows.

Dorchester Collection Conduit. This conduit, about 12 feet in size, would consolidate overflows 49, 50 and 67 and divert to a junction with the South Boston Conduit and the existing Moon Island system.

South Boston Collection Conduit. Overflows 76, 58, 42, 37, 14, 41, 30, 38, 97 and 35, essentially consisting of Reserved Channel and South Boston Beach area discharges, would be collected by this conduit. Varying from approximately 7 to 14 feet in size, the conduit would generally follow the shoreline to the Columbia Point area.

Columbia Point Tank. The Columbia Point tank would only receive flows in excess of the capacity of the Moon Island transport system. On this basis, such would generally not include first flush, but the more dilute extended flows. The tank would be about 1.7 cubic feet in size.

Dewatering of captured flow and wastes would be via diversion to the Columbia Park Headworks.

Moon Island Holding Facility. The existing basins on Moon Island would be upgraded and chlorination, screening and cleaning facilities would be installed. Dewatering would be as described in Alternative 2.

Other possible alternatives to this would be use of the Columbia Point Tank as a first flush capturing tank and the Moon Island system for extended flows disinfection. In this way, it may not be necessary to return flows from the Moon Island facility. Further, this alternative could be modified to retain overflows 37, 42, 58 and 76 in a tank in the Reserved Channel area and, thereby reduce flows to the Moon Island facility.

Calf Pasture Pumping Station. This pumping station would be upgraded with modern screening and pumping equipment to meet design flow conditions.

Outfalls. A cost allowance for outfalls has been made for both tanks. However, detailed circulation and water quality analyses are needed at both locations. It is expected that the present Moon Island surface discharge would be converted to a diffused bottom discharge in a location where benefit from depth and circulation will result.

Costs. A summary of major facilities along with their costs for Alternative 3 are shown in Table 5-5.

Cost Summary

The total construction and operation costs for each alternative are summarized in Table 5-6.

TABLE 5-6. SUMMARY OF CAPITAL AND OPERATION AND MAINTENANCE COSTS FOR COMBINED SEWER OVERFLOW REGULATION ALTERNATIVES

Alternative	Capital cost(1) (million dollars)	Operation and maintenance cost(1) (million dollars per year)
1	279	3.9
2	299	3.7
3	307	3.8

1. January 1975 costs (ENR 2200).

Costs for ongoing projects are not included such as separation in Cambridge and Somerville and the MDC Charles River Chlorination-Detention-Pumping Station Project.

TABLE 5-5. SUMMARY OF FACILITIES AND COSTS FOR ALTERNATIVE 3

Tank No.	Location	Outfalls to be collected	Tributary area,[2] acres	Tank size, (million cu ft)	Cost (million dollars) ENR 2200: Tanks	Pump station	Conduits	Total[1] plus 25% E&C
1	Brighton	10, 12	875	0.75	13.5	10.0	0.7	24.2
2	Brookline	13, 17	980	0.38	8.5	7.6	3.6	19.7
3	Back Bay Fens	16	7,550	0.75	15.3	18.3	2.5	36.1
4	Chelsea	21,63,31	330	0.18	4.4	3.3	2.9	10.6
5	East Boston (North)	34,32,3,2,4, 6,8	400	0.19	4.5	4.1	8.0	16.6
6	East Boston (South)	18,20,22,24, 26	525	0.24	5.6	4.8	3.9	14.3
7	Fort Point Channel	57,45,43,19, 9,27,33 2,960	2,960	1.00	15.0	20.0	4.8	44.8
10	Granite Avenue	54,51,53	720	0.40	9.0	9.0	4.3	22.3
12	Modified Moon Island Plan Columbia Point and Moon Island	76,58,42,37, 14,41,30,38, 97,35,28,67, 50,49	2,655		16.0[3]	21.9	67.1	105.0
	Subtotal							293.6
	Remote area projects							13.3
	Total							306.9

1. January 1975 prices (ENR 2200).
2. Includes separate areas that discharge into combined sewers.
3. Includes cost of a regulation facility at Columbia Point and rehabilitation of Moon Island facilities.

CHAPTER 6

EVALUATION OF ALTERNATIVES AND RECOMMENDED COURSE OF ACTION

Consideration of Water Quality Needs

Although the Boston Harbor and its tributary streams are one interrelated entirely, conditions and uses vary throughout. Recognizing this, the Massachusetts Division of Water Pollution Control has set differing standards in sections of the Boston Harbor area.*

In considering water quality problems and remedial needs related to combined sewer overflows, the areas tributary to sections of the Boston Harbor have been grouped as shown on Figure 4-1. General grouping are:

Dorchester Bay, including overflows from Dorchester and South Boston;

Charles River Basin, consisting of overflows upstream from the new Charles River Dam location, including existing regulation facilities;

Neponset River Estuary, including overflows from Dorchester; and

Inner Harbor, including main shipping areas of the Harbor and the estuary portions of the Charles and Mystic Rivers.

Dorchester Bay. This is the primary water contact recreation area in Boston Harbor with attendance well in excess of 10,000 persons per day. Its protection is of immediate importance and criteria used must relate to the objectives of maintaining water contact recreation there.

Charles River Basin. The Basin, with its shoreline parks, adjacent roadways and its bridges is the most visible water resource. Along with this is the high volume of small boat activity providing an even greater visibility to the public. Another critical resource in

*Boston Harbor Pollution Survey, Division of Water Pollution Control, Massachusetts Water Resources Commission, August 1970.

the Basin area receiving combined sewer overflows is the Back Bay Fens. It also is a high visibility resource.

In addition to the objectives of aesthetic quality, the Basin must be considered as a reservoir with a long residence time of pollution discharges to it. Settleable materials may even remain there permanently.

Regulation objectives of overflows in the Basin area must, therefore, include the removal of solids and floating matter along with an overall reduction of pollution discharges.

Neponset River Estuary. Third in priority for reduction of pollution from combined sewer overflows is the Neponset River area. Due to its potential effects on the beach and shellfish areas of Dorchester Bay and because of its classification as an area available for water contact recreation and restricted shellfishing, objectives must be addressed to those uses.

Inner Harbor. The Inner Harbor is considered lowest in priority of importance in remedial actions related to combined sewer overflows. Its classification will not permit its use for body contact recreation nor shellfishing. Since its use is primarily for commercial shipping and its shoreline is developed with piers and high walls, objectives of visual pollution abatement are most important. However, the potential effects of Inner Harbor discharges on the nearby beach areas must also be considered in deciding on solutions.

The combined sewer overflows in the Constitution Beach area are a special case in the Inner Harbor grouped overflows. Protection objectives there must be similar to those in Dorchester Bay.

Evalution of Alternatives

As stated in Chapter 4, good housekeeping is stressed here as an important contributor to water pollution control, but is not considered as an alternative. Also, source controls, such as ponding, are not considered feasible measures due to the highly developed combined sewer areas and because such do not provide positive measures at the outfalls once overflow does occur.

The major alternatives here are sewer separation, overflow diversions via Boston's proposed Deep Tunnel Plan and intermediate approaches of decentralized nature.

Sewer Separation. Sewer separation solely on the basis of water quality improvement is not recommended unless such is intended for the more remote areas. This is because of cost and the comparatively small water quality gains. However, it should be considered as a viable alternative in remote locations or where major improvements to the system are required in any event due to flooding problems. In such cases, however, the ensuing separated drainage system should incorporate measures for removal of debris, large solids and floating matter and possibly provide for disinfection in certain cases prior to discharge.

Sewer separation incorporating the above requirements is recommended in certain scattered locations as discussed in Chapter 5.

Deep Tunnel Plan. The Deep Tunnel Plan as proposed to the City of Boston is intended for complete diversion from the Harbor area of nearly all runoff reaching the combined sewer systems up to a 15-year design storm. However, a 1-year design storm is used as the basis for sizing decentralized system facilities in this study.

From a treatment point of view, removal of floatable matter would be required even for a deep ocean discharge as stated in Chapter 5.

In addition to requiring a total early commitment to this alternative, abandonment would be required of certain facilities that exist and others that are presently under construction.

The estimated cost in 1967 (ENR 1100 as opposed to ENR 2200 in this report) for the proposed Deep Tunnel Plan was about $430 million for construction and $2.63 million per year for operation and maintenance. As shown in Table 5-6, the estimated cost for additional facilities for a decentralized plan would be about $300 million and $3.7 million per year, respectively, for construction and operation and maintenance. To equitably compare the decentralized plan with the Deep Tunnel Plan the operation and maintenance costs for existing facilities were added to the $3.7 million cost for the decentralized plan. The cost for the Deep Tunnel Plan was adjusted by a simple ENR ratio to an ENR of 2200. Comparison of capital and operation and maintenance costs indicates that the decentralized plan is considerably less expensive than the Deep Tunnel Plan.

Another factor of importance is considering the Deep Tunnel Plan is the effect such a diversion will have on flushing action of the Harbor. Stormwater runoff provides occasional advective transport capability to the Harbor waters. Harbor water exchange with the ocean occurs as a result of circulation, diffusion and advective through flow. Other contributors to advection in the Boston Harbor are river flows and treatment plant discharges.

Decentralized Plan. As presented in this report, additional feasible alternatives exist in the form of decentralized solutions.

A decentralized plan would continue present remedial practices.

Such a plan would allow peacemeal implementation with immediate opportunities for solving high priority problem areas.

The degree and nature of the improvements could be geared to specific needs of each location and the extent of regulation provided could be carried out in stages so that advantage can be taken of evolving technologies in combined sewer overflow regulation and treatment. Treatment options under research and development have centered around physical treatment concepts of concentration, screening, sedimentation, flotation, filtration and disinfection. Nonmechanical concentration devices using induced fluid motions to separate settleable and floatable solids from overflows have shown solids removals in excess of 35 percent. High rate screening devices have received considerable attention, but present results have been disappointing. Sedimentation generally has been a secondary benefit to detention facilities. As shown in Chapter 5, results at the Cottage Farm facility have shown significant reductions due to sedimentation and capture. Due to objectives of aesthetic quality, dissolved air flotation has received considerable attention. A test facility in Wisconsin has shown average biochemical oxygen demand (BOD) and suspended solids (SS) removals at about 50 percent. Pilot plant results on high rate filtration have shown high BOD and SS removals. In addition, high removals of phosphates have been found. Recognizing the large storage capacity needed for disinfection contact, considerable effort has been addressed to high initial mixing concepts. Pilot plant work has demonstrated that disinfection at contact times as low as 1 to 2 minutes could be achieved.

In spite of these findings, the largest benefits in pollution reduction in decentralized systems will probably come from first flush capture and diversion to the dry weather flow treatment plant and through sedimentation, skimming and disinfection as a result of detaining overflows, while other treatment processes will be employed where such prove to be necessary for further polishing.

A large drawback in decentralized systems has been space requirement in high density land use areas. Some of this is being overcome by designs involving multiple use of land. For example, placement of overflow regulation facilities under parking garages, recreational facilities, parks, bus stops and the like is being practiced.

Another key factor in the decentralized approach is the selection of overflow groupings and the selection of overflow regulation facility discharge points. In the Charles River Basin area, overflow discharge concentration is dictated to some extent by overflow conduit arrangements and capacities originally constructed for the abatement of overflows there. The prime objectives in this location would be to make maximum use of existing facilities and provide treatment levels necessary. In the Back Bay Fens improvement of circulation would be an added objective.

Opportunities for alternative arrangements exist in the most critical water quality problem areas in Dorchester Bay and the Inner Harbor. Water quality considerations are discussed in the next section.

Boston Harbor Modeling Results

The Massachusetts Division of Water Pollution Control conducted water quality simulations for selected conditions using their Hydrodynamic and Time Variable Water Quality Models.*

Initially, output from the combined sewer modeling under existing conditions was simulated as discharged loads in terms of coliform bacteria. An attempt was made

*Development of Hydrodynamic and Time Variable Water Quality Models of Boston Harbor for Commonwealth of Massachusetts Water Resources Commission, Hydroscience, Inc., July 1973.

to compare these to measured coliform conditions resulting from a rainstorm in 1972. However, in the Inner Harbor area and near the Deer Island Treatment Plant measured discharge coliform levels were considerably higher than those simulated. Due to an overwhelming background level in the Inner Harbor area and the unknown disinfection conditions at Deer Island during this time along with the highly varying tidal conditions, conclusions could not be made without extensive further studies.

Since it is intended to destruct bacteria and remove floatable materials and large solids at the regulation facilities, the objective in evaluating alternative arrangements was to study the movement of simulated overflow volumes under design storm conditions which then can be extrapolated to concentrations of waste parameters as a measure of selecting discharge locations. Figures 6-1 through 6-4 give a picture of the results. The values recorded on these figures represent a comparison among the volume of overflow water discharged during a one-year, six-hour design storm under existing conditions and for the three decentralized alternatives described in Chapter 5.

The values shown are in parts of discharge water to million parts of Harbor water during high tide at 18.7 hours after the start of overflows. For example, the line showing 1,000 parts per million (ppm) represents a one tenth of a percent discharge water concentration. An overflow pollution concentration of 100 ppm (as for example for BOD) would convert to a concentration of one thousandth of a percent, or 0.1 ppm at that location. Present BOD background levels in the Harbor are about 1.7 ppm. It must be noted, however, that these are sectionally averaged values. Also, considerably more analysis and detailed testing and verification are required in order to take full advantage of this modeling as an aid in decision making for final design.

Under existing overflow conditions, the highest discharge volumes experienced, as shown on Figure 6-1 are at the mouth of the Charles River Estuary, in the Fort Point Channel, in the Old Harbor areas and in the Dorchester Bay area near Malibu Beach. Elimination of the effects in the latter two locations are of prime importance and are addressed by the decentralized alternatives discussed earlier.

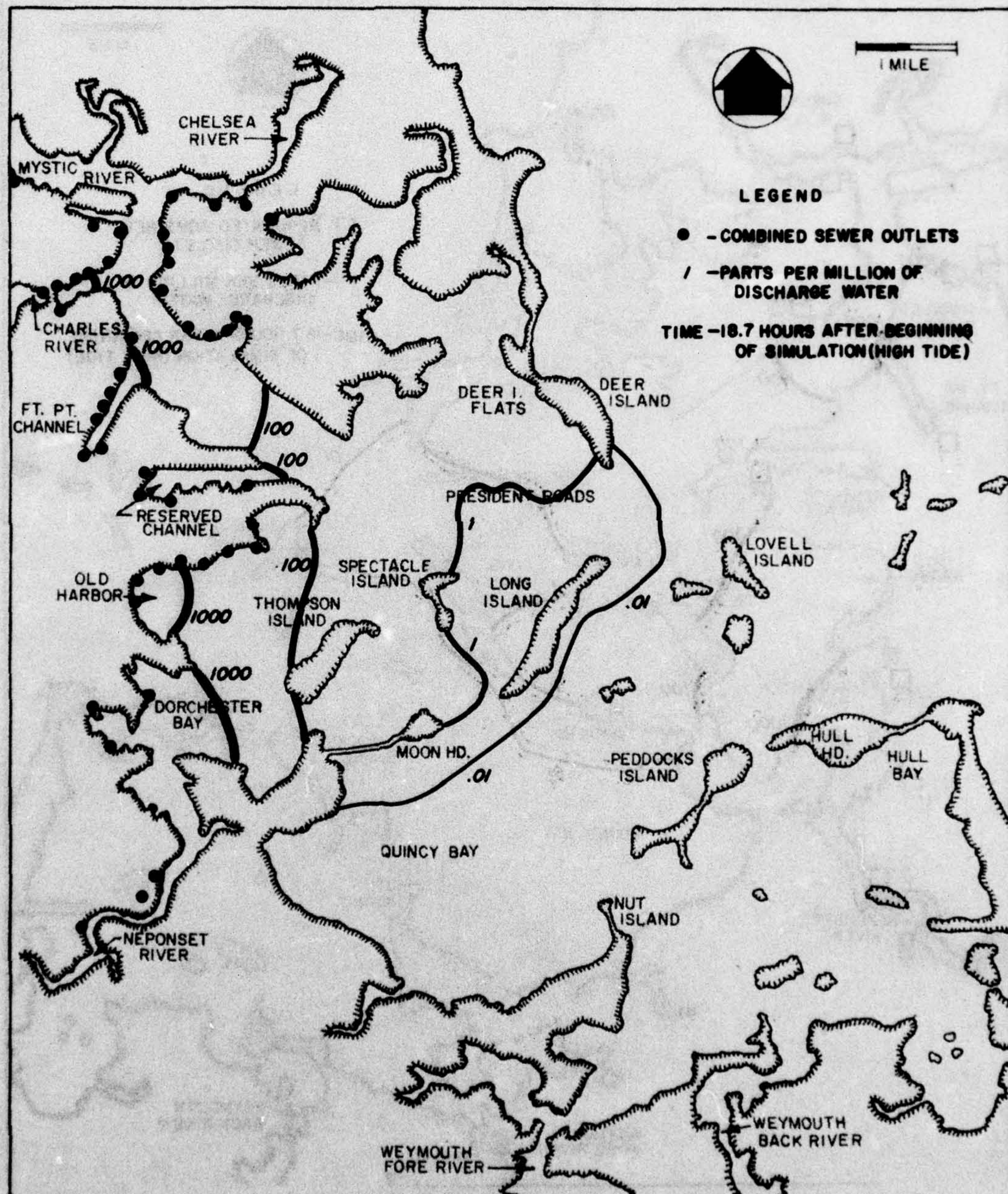

FIG. 6-1 HYDRODYNAMIC AND WATER QUALITY MODEL RESULTS IN BOSTON HARBOR FOR EXISTING CONDITIONS

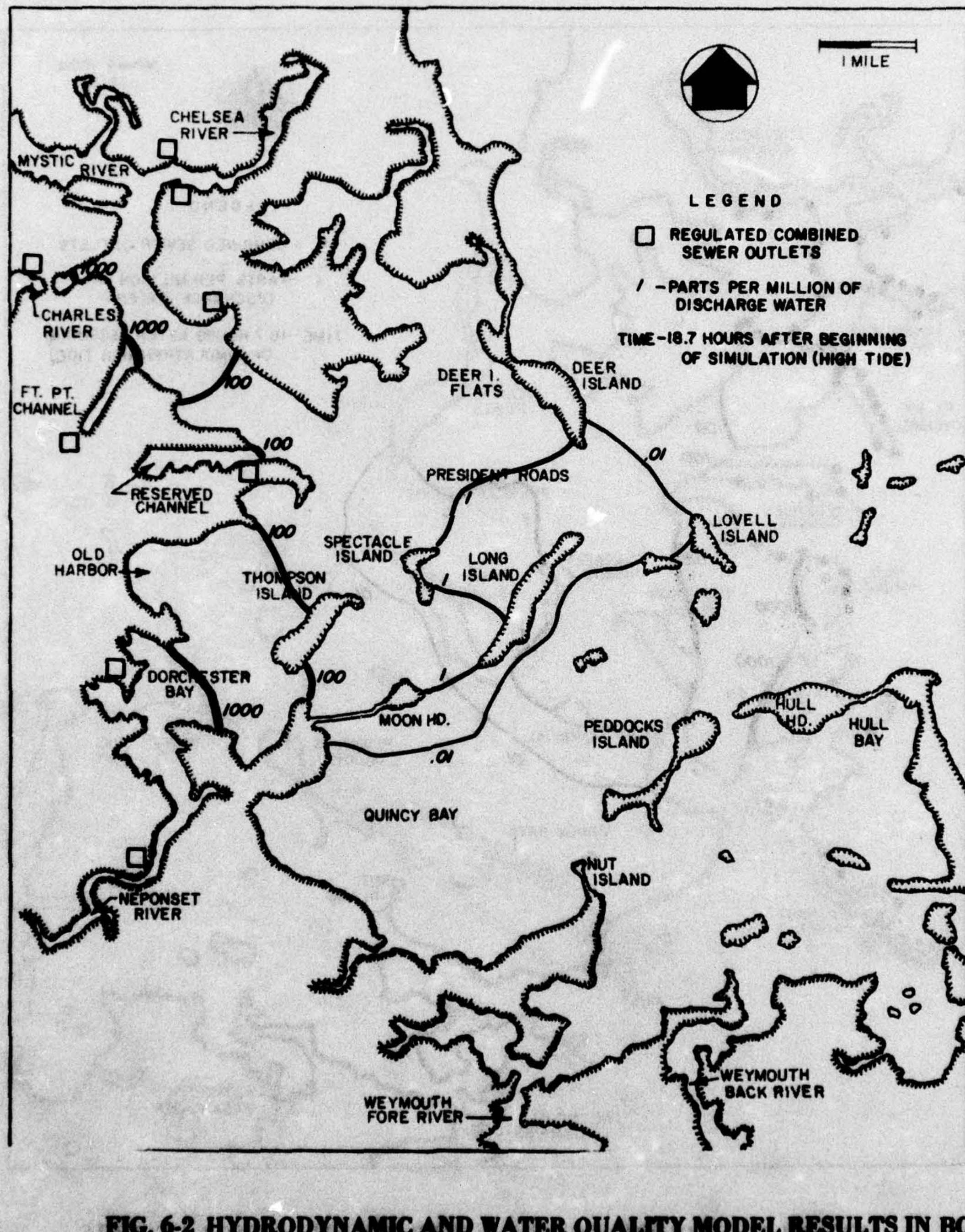

FIG. 6-2 HYDRODYNAMIC AND WATER QUALITY MODEL RESULTS IN BOSTON HARBOR FOR ALTERNATIVE 1 SATELLITE REGULATION FACILITIES

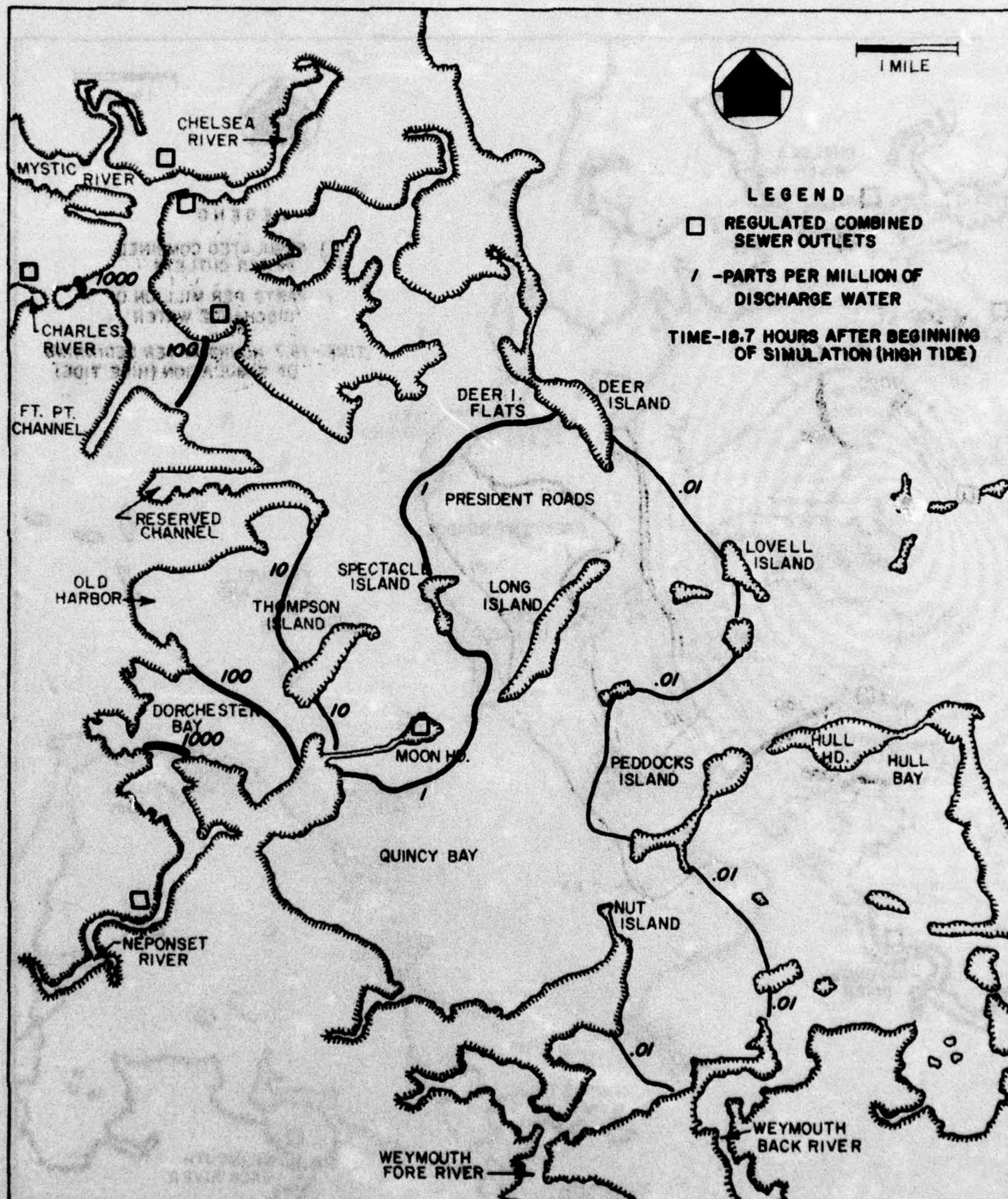

FIG. 6-3 HYDRODYNAMIC AND WATER QUALITY MODEL RESULTS IN BOSTON HARBOR FOR ALTERNATIVE 2 MOON ISLAND TUNNEL PLAN

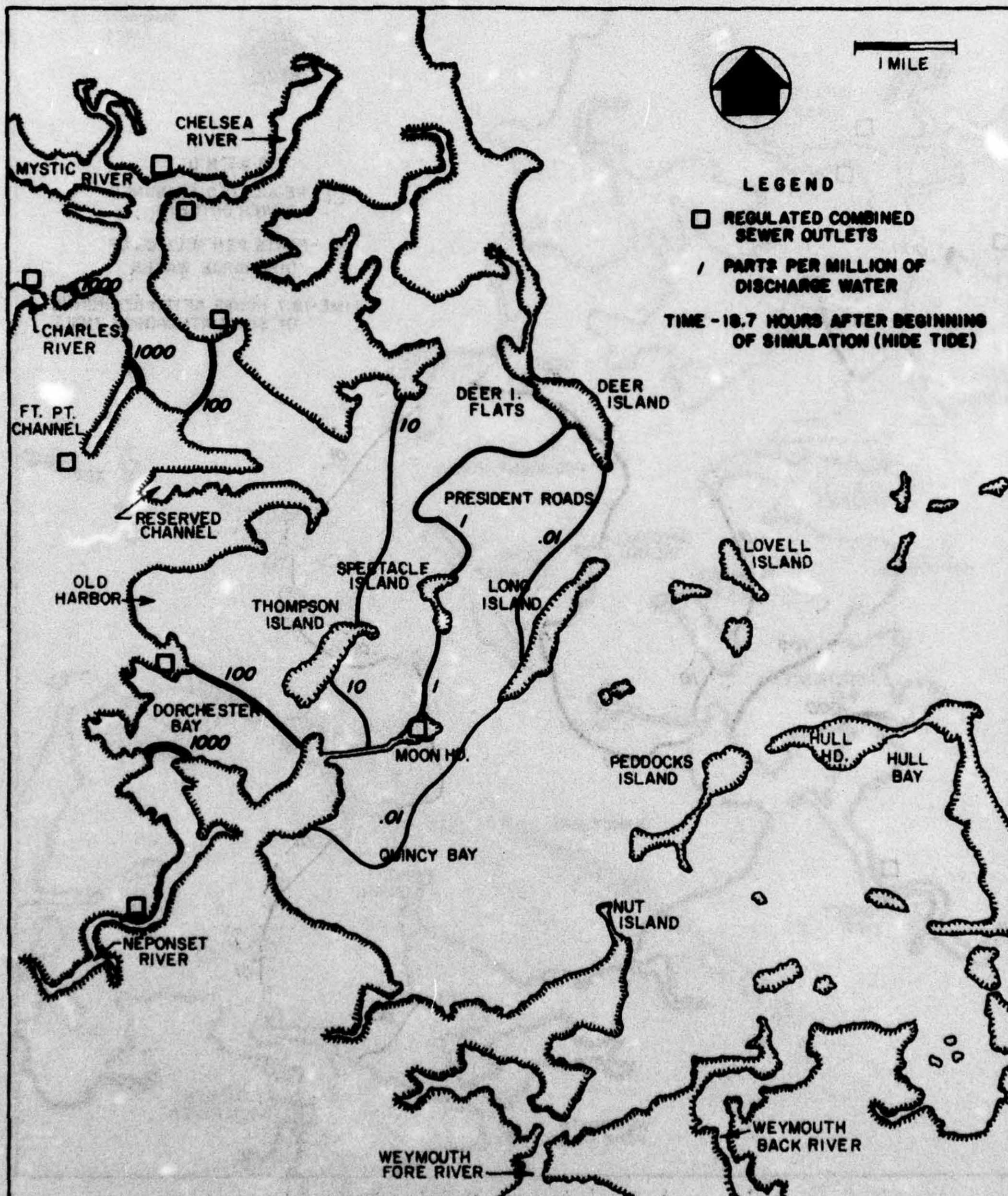

FIG. 6-4 HYDRODYNAMIC AND WATER QUALITY MODEL RESULTS IN BOSTON HARBOR FOR ALTERNATIVE 3 MODIFIED MOON ISLAND PLAN

Inspection of Figures 6-2 through 6-4 indicates that the simulations shows Alternative 2 to be slightly better than Alternative 3 and considerably better than Alternative 1 in Dorchester Bay. Alternative 1 consists of 10 regulation facilities and outfalls. Alternative 2 would route all tributary flow from downtown Boston, South Boston and Dorchester through the Moon Island facilities. Alternative 3 was operated on the basis of maximum use of Moon Island through the existing Dorchester Bay Tunnel with the Columbia Point regulation facility taking only excess flows.

In both Alternatives 2 and 3 discharge from Moon Island was from the existing shoreline outlets. Due to the higher flows diverted to this location in Alternative 2, effects from overflows increased into Quincy Bay. Special simulations evaluating the relocation of this outlet to deeper water near President Roads eliminated this effect on Quincy Bay.

An observation of the simulation of discharges into the Reserved Channel area indicates that such flows tend to creep into Dorchester Bay due to their proximity.

In general, these simulations indicate that overflow concentrations are small, if properly diffused; that overflows from the Dorchester Bay and other beach areas should be excluded; and that consideration should be given to retaining the Moon Island facilities, in some form, for purposes of combined sewer overflow regulation. Since the discharge system at Moon Island is to the water surface and, therefore, must be changed to a submerged and diffused system in an upgraded situation, the location of such discharge towards President Roads should be studied.

In further studies consolidation of combined sewer overflows from the Dorchester Bay area for discharges to the Inner Harbor and President Roads through Moon Island should be evaluated. However, it appears that diversion of overflows toward the Neponset River Estuary are not a beneficial approach.

It must be noted that these simulations represent a measure of overflows and associated wastes that mix and disperse in the water body. The movement of floatables, however, cannot be represented by such analysis. In addition, the study is preliminary and more detailed analyses are required for design decisions.

Recommended Course of Action

The recommended course of action is made on the basis that treatment will be extended to secondary at the Deer Island and Nut Island treatment plants. Should another alternative involving ocean discharge become the selected plan for these treatment plants, additional feasible opportunities to combined sewer overflow regulation in conjunction with dry weather flow treatment may become available.

The following course of action presents outline plans of study for facilities planning projects involving combined sewer overflow regulation.

Dorchester Bay Combined Sewer Overflow Regulation Project. This project would be for a facilities plan on the regulation of overflows in the Dorchester Bay area and should include:

1. Refinement of the combined sewer system models to increase confidence in the predictive ability of the model. As more field data is collected in the combined sewer system and the receiving waters parameters originally estimated based on limited data could be fine tuned and the model rerun to verify original estimates of flows and pollutants. The updated flow and pollution data could then be used to develop optimum design for transport conduits and related pollution control facilities.

2. Rainfall-runoff-overflow measurements in a selected controlled test area for model verification and parameter correlation. These measurements should extend into the receiving water.

3. Detailed consideration of special pollution sources, such as hospitals.

4. Refinement and verification of Harbor water quality simulation models for evaluation of potential discharge locations.

5. Evaluation of alternatives. Consideration of diverting discharges in the direction of the Neponset River Estuary do not appear desirable. However, alternatives of discharges in the Inner Harbor area and around Moon Island in the direction of President Roads should be investigated. Alternatives should be evaluated on

their performance over a longer hydrologic record under varied storm conditions so that appropriate design hydrology can be used in each case.

The period of record over which an alternative can be analyzed is a factor of the available rainfall data (and in some cases water quality data if water quality analysis is included) and the cost of the analysis technique. The actual period of time would vary on a case by case basis and is judgmental in nature but certainly should include what is by local standards a "wet year".

6. Detailed inventory and evaluation of the feasibility of upgrading the Moon Island facilities.

7. Site selection and preliminary engineering.

8. Consideration of multipurpose uses of land.

Charles River Basin Combined Sewer Overflow Regulation Project. This project should involve evaluation of the entire system related to combined sewer overflows tributary to the Basin once the New Dam and related facilities are completed. Included should be the Back Bay Fens area and the as yet unconnected overflows along the Charles River Basin. Facilities planning should emphasize an operating system towards optimum use of existing facilities along with treatment required at new facilities. The major project tasks should be:

1. Refinement of combined sewer models to the extent necessary so that all existing overflow conduits can be evaluated in detail.

2. Rainfall-runoff-overflow measurements in a selected controlled test area for model verification and parameter selection. Since the Basin essentially acts as a reservoir, exclusion of pollutants should be the objective rather than searching for an optimum discharge point.

3. Consideration of the state-of-the-art in storage-treatment concepts for overflows discharged above the new Charles River Dam.

4. Consideration of new regulator technologies for upgrading such facilities at the existing overflow conduits.

5. Evaluation of alternatives. Optimum solutions in this project area appear to be an operating system that would make maximum use of existing facilities in such a way that first flush effects are transported to facilities below the Dam for treatment and discharge, or are stored and treated more extensively prior to discharge into the Basin, or are stored and diverted to the Deer Island Treatment Plant. Performance of alternatives under longer term hydrologic records must be part of the evaluation. In the development of alternatives, unconnected overflows must be included. Similarly, existing overflow conduits should become part of the operating system.

6. Incorporation of Back Bay Fens recreation objectives in plan selection. In the development of alternatives, the problems and objectives of the Back Bay Fens water resource should be incorporated into the project. For example, solving the Fens circulation problems should be part of the objectives of combined sewer overflow regulation there.

7. Site selection and preliminary engineering.

8. Consideration of multipurpose use of land. In this case, multiuse alternatives would be especially important due to the high recreational potentials in the Back Bay Fens and along the Basin.

Neponset River Combined Sewer Overflow Regulation Project. Due to its location, alternatives in this project area would primarily address the search for a cost effective solution to minimize pollution discharges and the site selection alternatives for appropriate regulation facilities. The project tasks should include:

1. Refinement of the combined sewer system models.

2. Evaluation of alternatives. Again, performance on the basis of longer range hydrologic data should be evaluated.

3. Site selection and preliminary engineering.

Inner Harbor Combined Sewer Overflow Regulation Project. It appears that consolidation of overflows in the Inner Harbor area will be primarily directed at overcoming constraints associated with space needed for conduits and regulation facilities. Therefore, primary efforts in this area should be directed at the technical problems of conduit location, regulator design and discharge pipe location. The facilities plan should cover among other things the following:

1. Refinement of combined sewer system models.

2. Detailed consideration of industrial pollution sources.

3. Evaluation of consolidation alternatives based on the technical problems stated above.

4. Site selection and preliminary engineering.

5. Consideration of multipurpose use of land.

6. Evaluation of overflows in the Constitution Beach area as a special case.

Special Projects. The combined sewer overflows not tributary to a regulation facility should be evaluated in accordance with possible solutions as listed in Table 5-2.

Other special studies, as mentioned under several of the above projects, should be sample area monitoring of the rainfall-runoff-overflow process. Evaluation of such for purposes of verifying and modifying parameters for combined sewer overflow simulation should be carried out. Similarly, in the case of overflows in the beach areas, further detailed studies of that Boston Harbor area receiving water should be carried out to aid in selection of optimum discharge locations.

APPENDIX A

HYETOGRAPH DEVELOPMENT

APPENDIX A

HYETOGRAPH DEVELOPMENT

Development Procedure

In using simulation as a technique for flow determination, time-varying parameters are required. On this basis, rainfall data representative of actual conditions are necessary rather than using time-averaged rainfall values which are then multiplied by a constant to obtain peak runoff as is done in the Rational Method.

The steps used in developing rainfalls for the design storms is as follows:

1. Select the desired range of frequencies and durations for the storms, i.e., 1-, 2-, 5-, or 15-year storms and 2- or 6-hour durations. A duration of 6 hours was found to be the average duration for storms in the Boston area from studying records of the 1962-72 period.

2. Using the U. S. Weather Bureau data*, select the appropriate isopluvial maps corresponding to the particular storm frequencies and durations. Locate the study area on the maps and record the rainfall depths, interpolating between isopluvial lines. This rainfall depth, and frequency information for Boston Metropolitan area is presented in Table A-1. Also shown are the average rainfall intensities corresponding to the depths and durations. For rainfall durations less than 30 minutes, isopluvial maps do not exist. However, the U. S. Weather Bureau* provides constants for converting rainfall depths at 30-minute durations to depths at 5-, 10-, and 15-minute durations.

3. Select a simulation time increment for each storm duration. A 10-minute increment was chosen for the 6-hour storm. This was judged to best describe the storm characteristics and not consume large amounts of computer time.

*U. S. Weather Bureau, <u>Rainfall Frequency Atlas of the United States</u> for durations from 30 minutes to 24 hours and return periods from 1 to 100 years, Technical Paper No. 40, Washington, D. C., May 1961.

TABLE A-1. RAINFALL INTENSITY-DURATION RELATIONSHIPS FOR FREQUENCIES OF 1, 2, 5 AND 15 YEARS IN BOSTON, MASSACHUSETTS

	Rainfall depths in inches and intensities in inches per hour for various frequencies							
	1 year		2 years		5 years		15 years	
Duration	in.	in./hr	in.	in./hr	in.	in./hr	in.	in./hr
5 min	.285	3.42	.348	4.18	.463	5.56	0.58	6.96
10 min	.438	2.63	.535	3.21	.713	4.28	0.89	5.35
15 min	.555	2.22	.677	2.71	.900	3.60	1.13	4.52
30 min	.770	1.54	.94	1.88	1.25	2.50	1.55	3.10
1 hr	.98	.98	1.17	1.17	1.57	1.57	1.96	1.96
2 hr	1.26	.63	1.52	0.76	2.00	1.00	2.54	1.27
3 hr	1.41	.47	1.80	0.60	2.32	0.77	2.82	0.94
6 hr	1.78	.30	2.30	0.383	2.80	0.467	3.42	0.57

4. Select a storm pattern. Special consideration must be given to locating the rainfall peak at a point from the beginning of the storm that is representative of storms in the Boston area. Figure A-1, which shows the average distribution of rainfall and the time to maximum intensity, was developed from selected rainfall records at Logan Airport, Boston, covering the period 1960 to 1972. Figure A-1 was developed from 11 selected storms ranging from 5 to 9 hours in duration and serves as the basis for locating the maximum rainfall intensity in the 6-hour design storm.

5. The design hyetographs for a storm of a given frequency and duration were prepared as follows:

   a. A bar graph was drawn with TIME in minutes plotted on the x-axis and RAINFALL DEPTH in inches plotted on the y-axis.

   b. The point in time was located where the maximum rainfall intensity will occur. For a 6-hour storm duration, this point occurs 165 minutes after start of storm according to Figure A-1.

   c. At the point of maximum intensity, the rainfall depth corresponding to the selected time increment was plotted. For a 6-hour storm with a 10-minute time increment, the 1-year rainfall depth during the maximum 10-minute interval is 0.438 inches, while the corresponding rainfall depth for a 15-year frequency storm is 0.89 inches. (See Table A-1).

   d. Using the location of the hyetograph peak as a starting point, rainfall depths were plotted for each time increment corresponding to the storm patterns indicated on Figure A-1. The rainfall depth for any given time period equals the maximum depth indicated in Table A-1 for a similar period.

   e. The ordinate of the above graph was then converted to rainfall intensity in inches per hour to produce the design hyetographs.

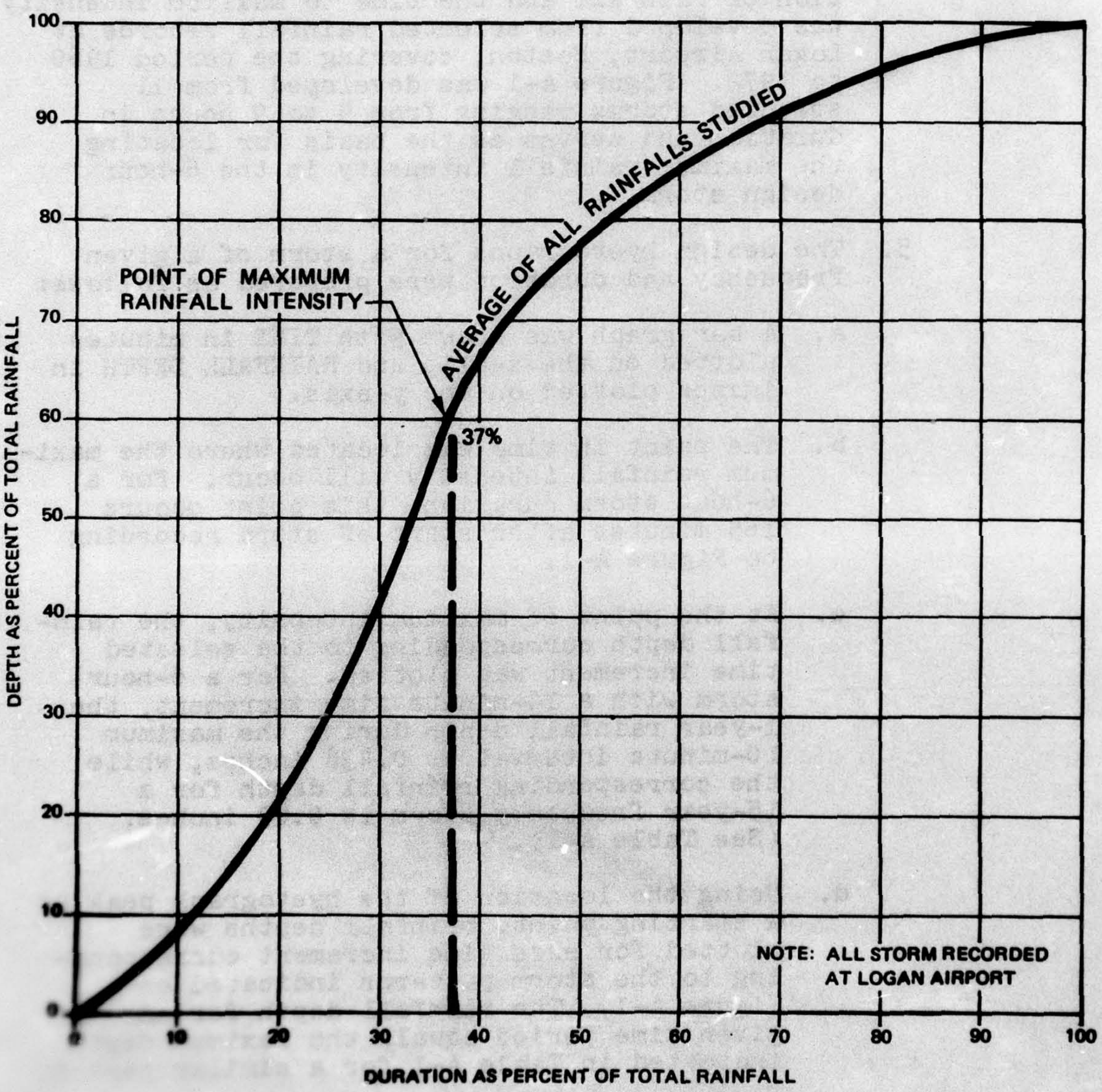

FIG. 4-1 RAINFALL MASS DIAGRAM BASED ON DATA FROM 11 RANDOMLY SELECTED STORMS FOR 5 TO 9 HOURS IN DURATION

Design Hyetographs

Figures 3-1 (in Chapter 3) and A-2 show the design hyetographs for 1- and 15-year, 6-hour duration design storms. It should be noted that these hyetographs do not represent actual storm events, but are synthetic storms fulfilling the U. S. Weather Bureau statistics on rainfall depth-duration-frequency and rainfall distribution pattern judged typical for the Boston area.

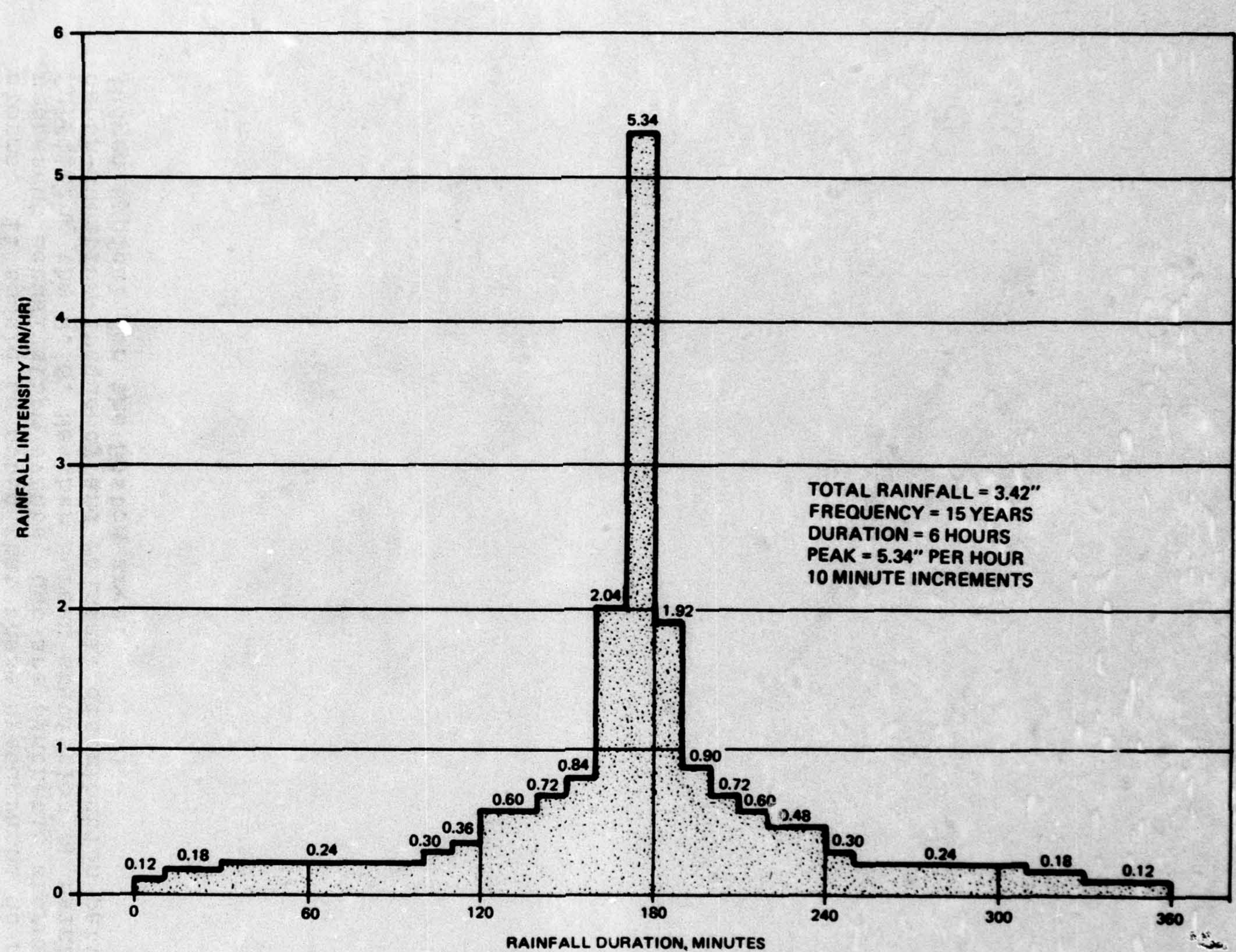

FIG. A-2 15-YEAR 6-HOUR DESIGN HYETOGRAPH

APPENDIX B

COMPARISON OF 1- AND 15-YEAR DESIGN STORMS

APPENDIX B

COMPARISON OF 1- AND 15-YEAR DESIGN STORMS

Although the scope of this study does not permit an in-depth analysis of hydrologic conditions to fine-tune the selection of design storms, a comparison was made of storms of various severities to demonstrate that the hydrologic design criteria for purposes of drainage protection and for water quality control should be different. Final determination of the degree of protection must, however, also be dependent on the receiving water quality analysis during detailed facilities planning.

Past experience indicates that for water quality control use of storms of 1- or 2-year return period is advisable. The three key reasons for using the shorter frequency storm for water quality control are as follows:

1. The pollution load discharged increases at a lesser rate than flow with increased storm severity as can be seen from Table B-1 below.

TABLE B-1. COMPARISON OF OVERFLOW FROM A 1- AND A 15-YEAR STORM AS SIMULATED AT THE LOWELL STREET OVERFLOW

Parameters considered	Storm severity 1 year	Storm severity 15 years	Ratio 15-year to 1-year storm
Rainfall, inches	1.78	3.42	1.92
Duration, hours	6	6	1.00
Peak flow, cfs	150	228	1.52
BOD_5, lb	366	484	1.32
SS, lb	5,630	7,528	1.33

2. The increase in the number of storms treated by going from a 1-year to a 15-year storm is minimal. Combined sewer overflow studies conducted by Metcalf & Eddy, Inc. in Middletown, Ohio, and in Washington, D. C. demonstrated this. It is

judged that conditions in the EMMA area are similar to those in Middletown and Washington, D. C. Table B-2 presents a summary of the Middletown and Washington, D. C. data.

TABLE B-2. STORM FREQUENCY VERSUS RETURN PERIODS BASED ON PEAK HOURLY INTENSITY

Parameter	Middletown, Ohio	Washington, D. C.
Period of record	1950 to 1972	1950 to 1971
Length of record, years	22	21
Total number of storms(1)	2,756	1,817
Storms smaller or equal to 1-year severity	2,724	1,800
Storms greater than 1-year severity(2)	32	17
Average number of storms per year with a peak hourly intensity greater than 1-year storm	1.45	0.81

1. A storm is defined as any measurable rainfall separated from other precipitation by at least 6 hours.
2. Severity is measured on the basis of peak hourly intensity during a storm.

3. The first flush action in the sewer system will enable a facility designed for a small frequency storm to catch and treat the bulk of the pollutants from a larger frequency storm. This can be seen on Figures B-1 and B-2 which plots computed hydrograph and pollutographs from the Lowell Street drainage area for the 15-year, 6-hour design storm. Flows up to the peak design flow of 150 cfs for the 1-year, 6-hour design will receive intended detention and chlorination. Flows greater than 150 cfs will receive reduced detention and chlorination in ratio of the 1-year design peak flow to the actual flow. However, this reduced detention and chlorination will be for a small volume of

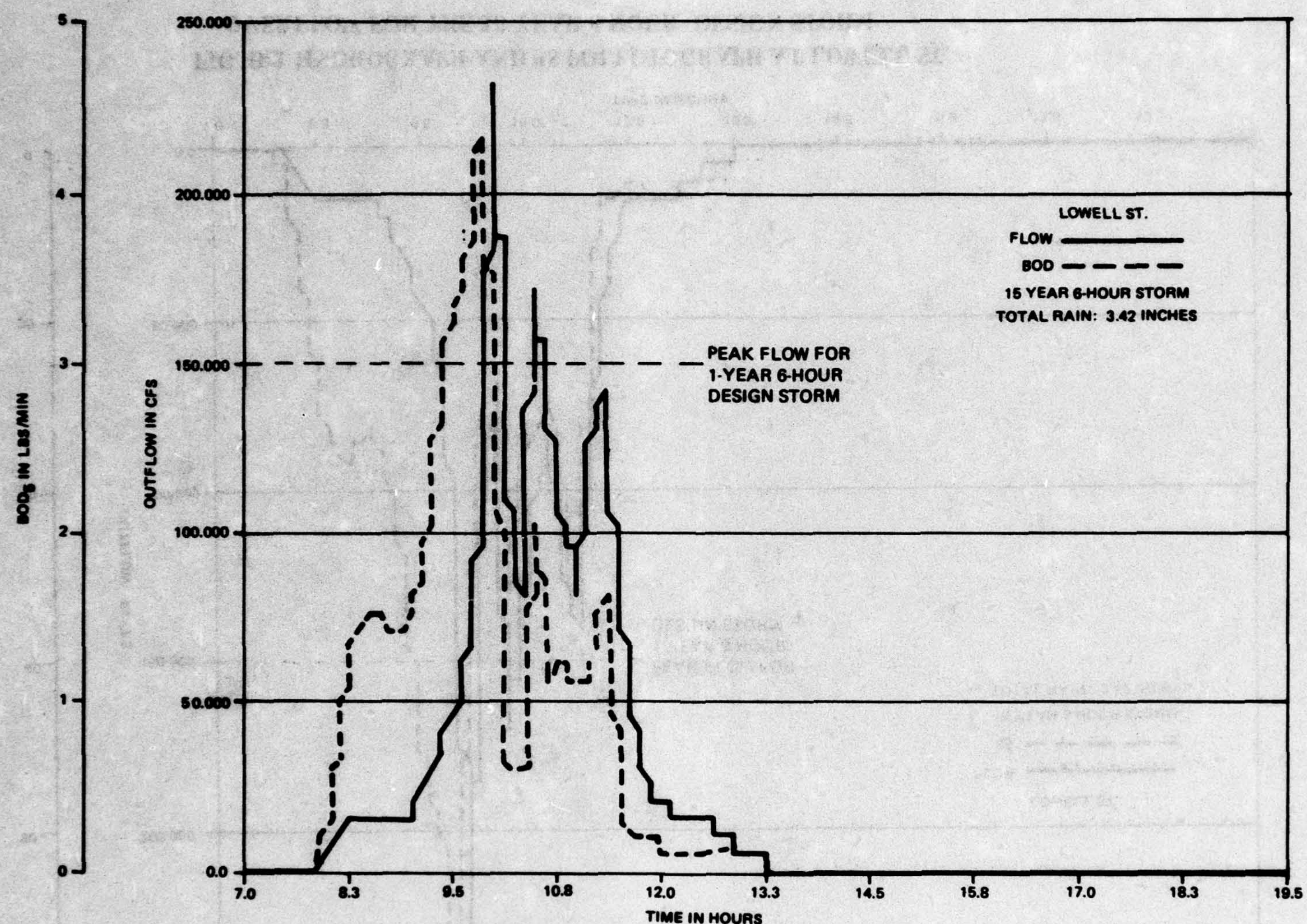

FIG. B-1 HYDROGRAPH AND BOD_5 POLLUTOGRAPH AT LOWELL ST. OVERFLOW FOR THE 15 YEAR 6-HOUR DESIGN STORM

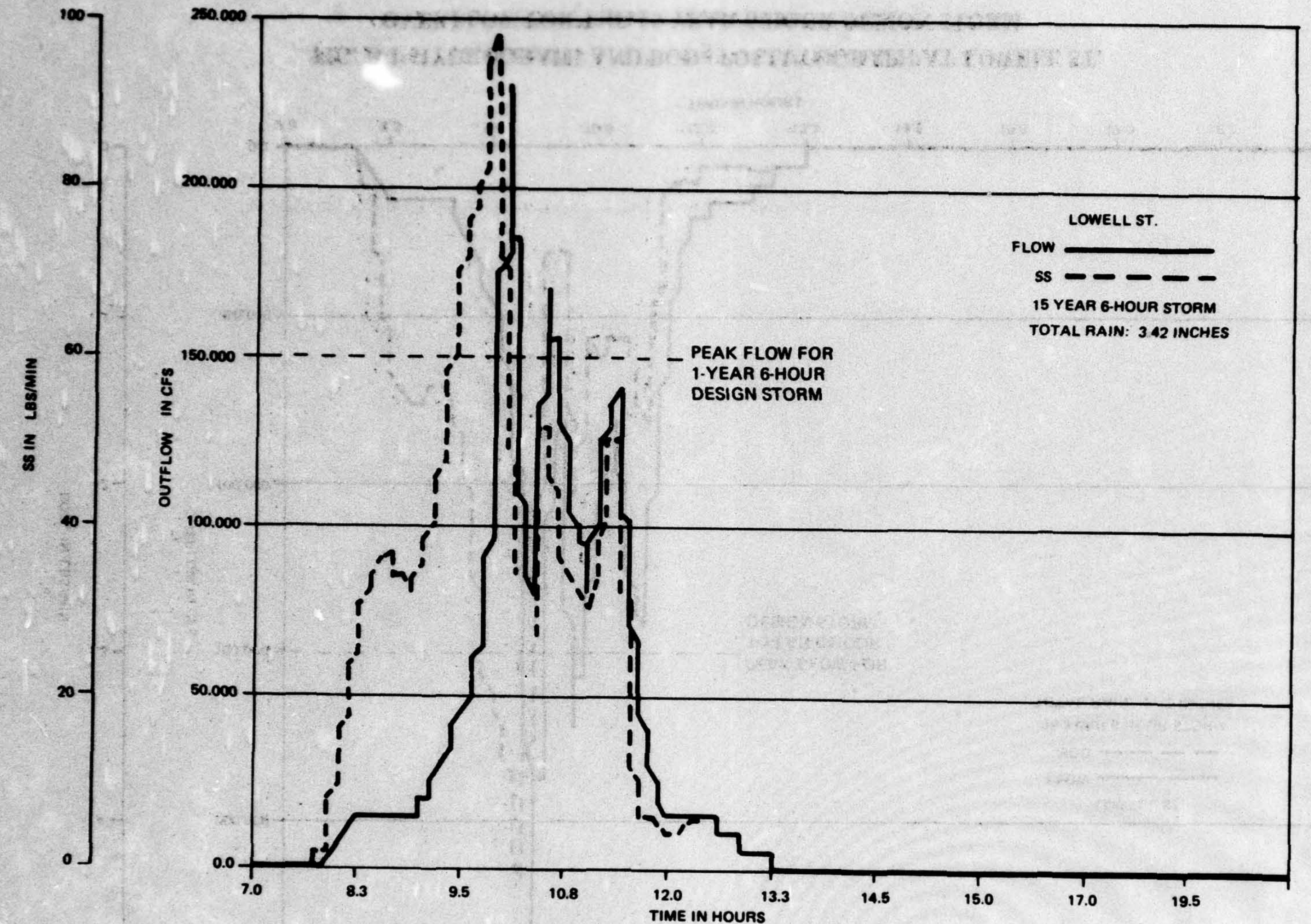

FIG. B-2 HYDROGRAPH AND SS POLLUTOGRAPH AT LOWELL ST. OVERFLOW FOR THE 15 YEAR 6-HOUR DESIGN STORM

the overflow compared to the total volume of the overflow as can be seen from Figures B-1 and B-2.

APPENDIX C

MODEL DEMONSTRATION AND APPLICABILITY

APPENDIX C

MODEL DEMONSTRATION AND APPLICABILITY

The purpose of demonstrating a model is to insure that the observed conditions in the study area are being represented by the mathematical formulations in the model to the extent necessary for decisionmaking. In order to demonstrate the Storm Water Management Model's (SWMM) applicability in the Boston Harbor combined sewer area measurements at the Lowell Street overflow* (area C-14 on Figure 1-1) were compared with values generated by the SWMM under similar conditions.

The Lowell Street drainage area is located on the eastern edge of Cambridge bordering the Charles River at its outfall. The drainage area tributary to the combined sewer system includes approximately 222 acres of predominantly residential land area with an average percent imperviousness of about 40 percent. The area, although partly separated, reacts during wet weather conditions as a combined system.

Presently the combined system is connected to the MDC North Charles Metropolitan Sewer at Mt. Auburn and Lowell Streets by a 24-inch connection at a regulator. Flow in excess of the regulator capacity is diverted through the 52-inch overflow pipe to the Charles River. It is the flow through the 52-inch pipe that has been measured and simulated. Upon completion of the North Charles Relief Sewer flow up to 10 mgd will be diverted into the relief sewer. Flow in excess of 10 mgd will overflow to the Charles River.

Since rainfall was not being measured at the Lowell Street overflow during these measurements, recorded hourly increments at Logan International Airport approximately 7 miles from the study area were selected. However, the recorded rainfall at Logan Airport may be different from the rainfall at the site due to aerial variability.

In order to reduce this uncertainty data from two other stations collecting daily records was compared to the daily records from Logan Airport during that period.

*Combined Sewer Overflows to the Charles River, Commonwealth of Massachusetts Water Resources Commission, by Process Research, Inc., 1972.

These two stations and the Logan Airport rain gage form a triangle which encloses the Lowell Street study area. These two stations are:

1. Chestnut Hill Reservoir (U. S. Weather Bureau)

2. Spot Pond in Stoneham (U. S. Weather Bureau)

For the storms of 25 November 1974 and 29-30 November 1974, used to simulate Lowell Street overflows, the correlation among the three stations is found to be very good ($\pm$ .01 inch total volume) indicating that the two storms recorded at Logan Airport are representative of the rainfall in the study area.

Figure C-1 shows the computed and observed hydrographs for the storms of 25 November 1971 and Figure C-2 shows the computed and observed hydrographs for the storm of 29 November 1971. Respective rainfall hyetographs are also shown on each figure.

<u>Storm of 25 November 1971</u>

<u>Storm characteristics</u>

Total rainfall	2.59 inches
Duration	19 hours

The shape of the observed and computed hydrographs for this storm conform well throughout the first 11 hours of the storm. At this time the observed discharge drops off sharply and then increases to a peak of 28 cfs at 14:15 hours. The computed discharge does not decrease after 11 hours but rather continues to increase to a peak discharge of 40.8 cfs at 14:15 hours. Since the recorded rainfall record indicates no marked reduction in rainfall after 11 hours, no reduction in discharge should be expected to occur. It is believed that the sudden reduction of the observed flow after 11 hours indicates one of two things:

1. There was an error due to malfunctioning of measuring equipment, or

2. The actual rainfall in the study area differed from the rainfall recorded at Logan Airport.

The observed hydrograph was adjusted to eliminate the sudden dip and the result shows better correlation with the computed values (dashed line on Figure C-1).

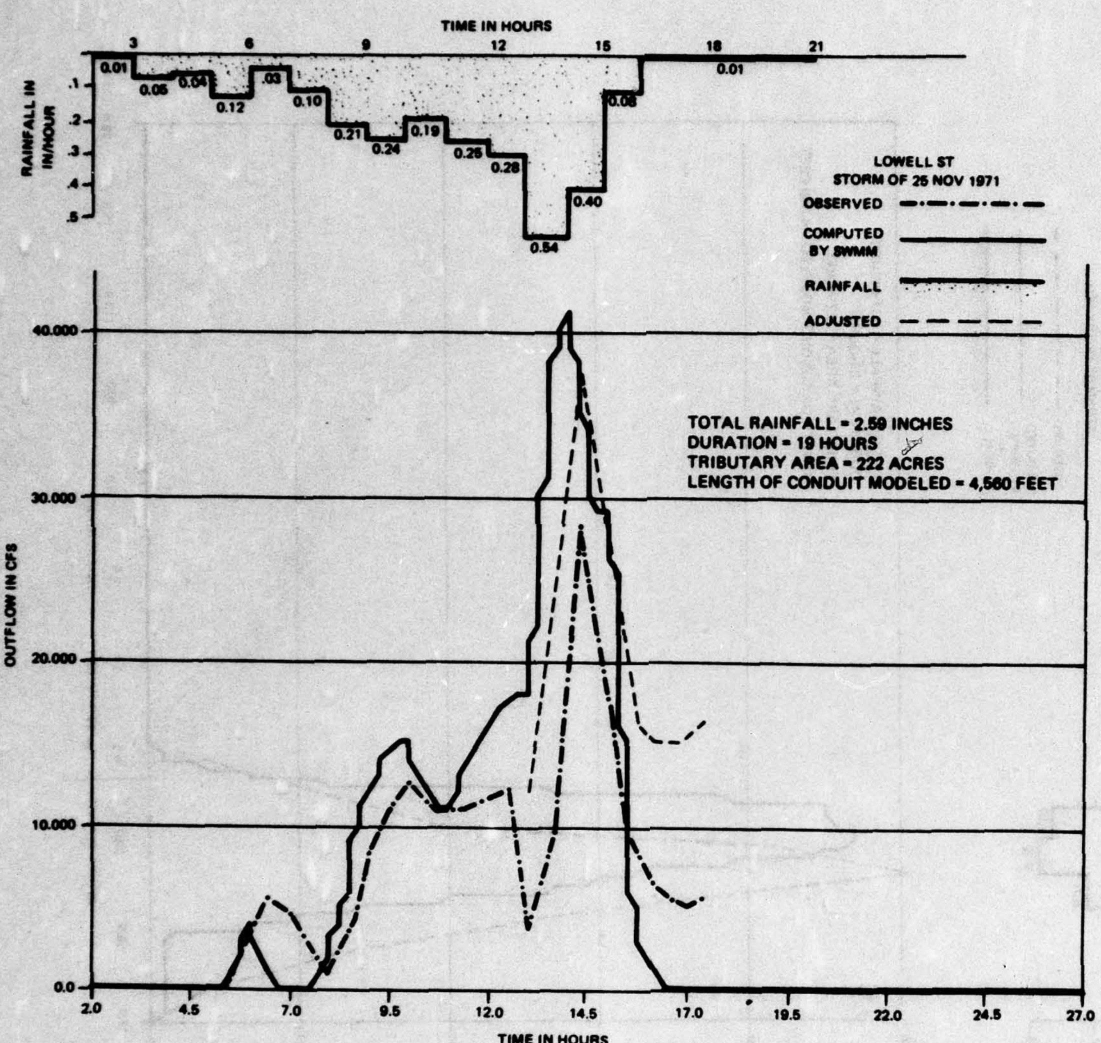

FIG. C-1 COMPARISON OF COMPUTED AND OBSERVED HYDROGRAPS AT LOWELL ST. OVERFLOW FOR THE STORM OF NOVEMBER 25, 1971

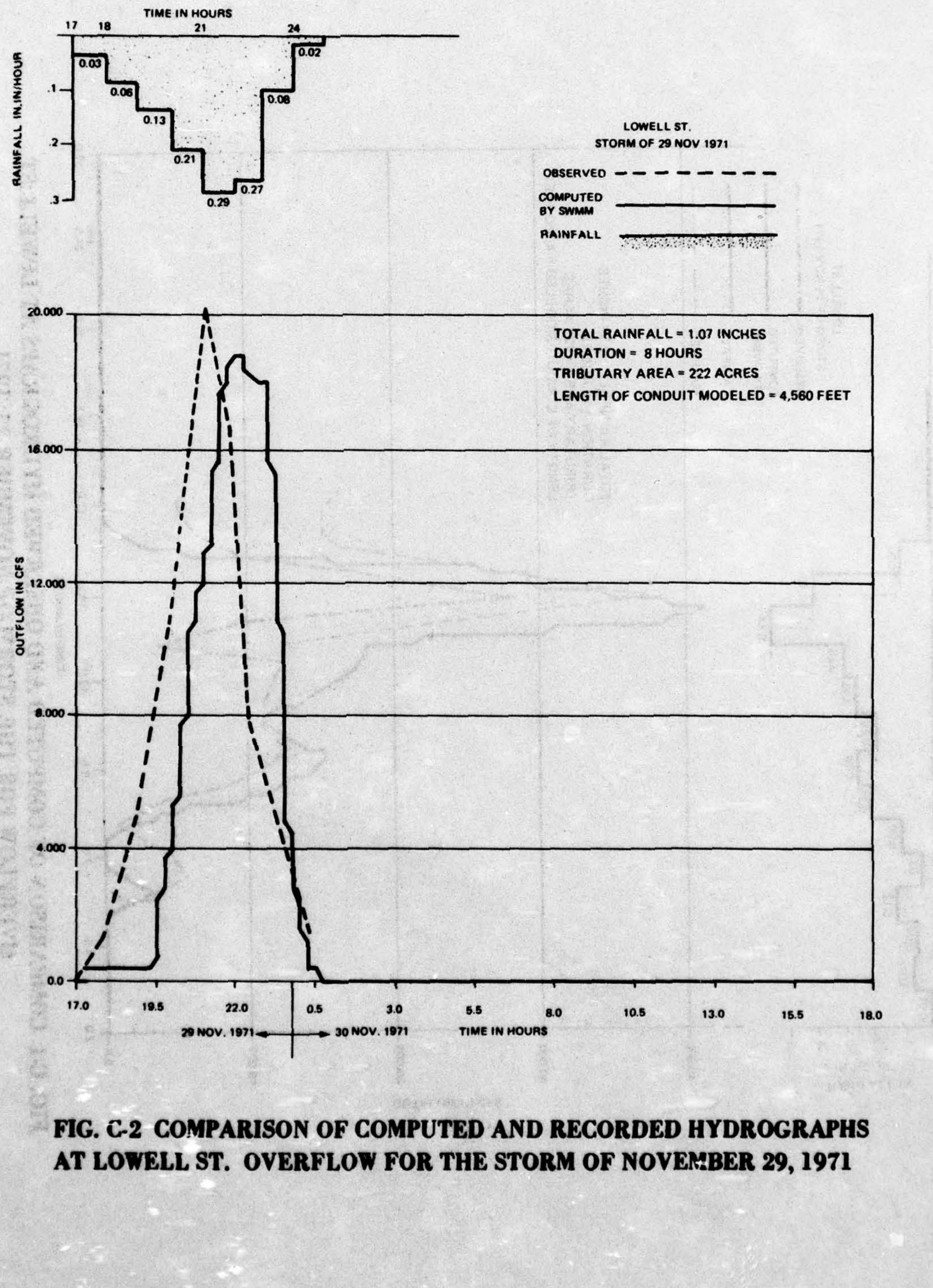

FIG. C-2 COMPARISON OF COMPUTED AND RECORDED HYDROGRAPHS AT LOWELL ST. OVERFLOW FOR THE STORM OF NOVEMBER 29, 1971

AD-A036 840 METCALF AND EDDY INC BOSTON MASS F/G 13/2
WASTEWATER ENGINEERING AND MANAGEMENT PLAN FOR BOSTON HARBOR - --ETC(U)
NOV 75

UNCLASSIFIED NL

2 OF 2
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Storm of 29-30 November 1971

Storm characteristics

Total rainfall	1.07 inches
Duration	6 hours

The hydrographs for the observed and computed discharges at Lowell Street for the storm of the 19-30 November 1971 compare very well. The shapes are very similar and the peak observed flow of 20 cfs differ by only 5 percent from the computed flow of 19 cfs.

APPENDIX D

SIMULATED FLOW AND POLLUTION CHARACTERISTICS OF DRAINAGE BASINS IN THE COMBINED SEWER STUDY AREA

APPENDIX D

SIMULATED FLOW AND POLLUTION CHARACTERISTICS OF DRAINAGE BASINS IN THE COMBINED SEWER STUDY AREA

The Simulated Flow and Pollution Characteristics of each Drainage Basin are divided into two tables. Table D-1 lists the flow, BOD_5 and SS for each Drainage Basin which contains outfalls to the Receiving Waters and serves to summarize the extent of the overflow problem under design conditions. Table D-2 lists Rainfall Runoff and Percent Runoff for each Drainage Basin.

TABLE D-1. OVERFLOW CHARACTERISTICS OF EACH DRAINAGE BASIN FROM 1-YEAR-6 HOUR DESIGN STORM

Municipality	Basin[1]	Outfall number(s)[2]	Peak flow[3], (cfs)	BOD_5, (lb)	SS, (lb)
Boston	B2	119,34,32, 2,3,4,6,8, 18,20,22, 24,26	670	1,800	28,800
Boston	B3	10,12,63	780	2,600	35,800
Boston	B4	17,16,40, 44,25,46, 48,15,52, 600	4,080	21,600	257,600
Boston	B5	57,45,43, 19,9,27	560	3,800	25,300
Boston	B9	33,35,28	1,700	11,600	112,300
Boston	B9A	95[4],76, 58,42	200	200	3,800
Boston	B9B	37,14,41, 30,38,97	330	1,200	13,100
Boston	B12	67,50,49	1,050	5,300	64,200
Boston	B16	54,51,53	650	2,300	21,800
Brookline	BR	13	480	5,500	35,600
Cambridge	C12	5	230	800	10,200
Cambridge	C14	7	150	400	5,600
Chelsea	CH	1,21,63,31	340	1,800	23,300
Somerville	S1	36	610	10,400	104,800
Somerville	S2	90,88[4]	110	4,200	32,000

1. B1 represents Charlestown which is separating its sewers and is, therefore, omitted from the combined pollution analysis.
2. For location see Figure 2-1.
3. Peak flow, BOD_5 and SS represent a simple addition of each parameter for all overflows in a drainage area.
4. Overflow does not discharge during a one year design storm.

TABLE D-2. RAINFALL AND RUNOFF VOLUMES FROM EACH DRAINAGE BASIN FOR A 1-YEAR-6-HOUR DESIGN STORM

Municipality	Basin[1]	Rainfall, (cu ft)[2]	Runoff, (cu ft)[2]	Percent runoff
Boston	B2	6,463,700	3,203,900	50
Boston	B3	8,369,200	4,466,100	53
Boston	B4	7,973,500	5,080,000	64
Boston	B5	5,394,100	3,858,900	72
Boston	B8[3]	15,673,300	8,867,700	57
Boston	B9[4]	15,316,500	7,072,800	46
Boston	B10	7,130,800	1,952,700	27
Boston	B11	14,840,000	5,620,500	38
Boston	B12	13,911,200	8,174,300	59
Boston	B15	25,186,300	10,040,100	40
Boston	B16	9,195,300	5,014,500	55
Brookline	BR	3,216,200	1,912,600	59
Chelsea	CH	2,975,900	2,054,700	69
Somerville	S1	8,850,800	5,012,600	57
Somerville	S2	2,733,400	1,820,800	67
Cambridge	C3	3,902,000	2,603,000	67
Cambridge	C7	647,900	346,300	53
Cambridge	C8	140,400	109,200	78
Cambridge	C9	885,400	686,400	77
Cambridge	C10	2,472,000	1,483,100	60
Cambridge	C12	1,423,200	909,300	64
Cambridge	C14	1,432,000	564,300	39
Cambridge	C16	763,900	519,700	68

1. Basin B1 represents the Charlestown section of Boston which is separating its sewers and is, therefore, not included in the combined sewer analysis.
2. Rainfall and runoff volumes are calculated in the Storm Water Management Model for each basin.
3. Basin B8 includes B8A and B8B.
4. Basin B9 includes B9A and B9B.

APPENDIX E

COMPUTER MODELING INSTRUCTIONS

APPENDIX E

COMPUTER MODELING INSTRUCTIONS

This appendix contains the user instructions for the computer programs necessary for the modeling and analysis of combined sewer systems. Included here is the EPART PROGRAM MANUAL.

EPART PROGRAM MANUAL

I. GENERAL DESCRIPTION

A. NAME: EPART

EPART is a modified version of certain parts of the EPA Storm Water Management Model (EPASWMM). It basically contains the EXECUTIVE, RUNOFF, TRANSPORT, and COMBINE blocks of EPASWMM with modifications to facilitate the use of the model for combined sewer system analysis. However, EPART has retained all capabilities of these blocks as they were developed originally for EPASWMM.

B. DESCRIPTION:

EPART is a comprehensive mathematical model capable of representing urban stormwater runoff and sewer flow phenomena. This mathematical model is used to simulate storm events on the basis of rainfall inputs, (hyetographs) and system characterization (overland and sewer flow) to predict outcomes in the form of time varying quantity (hydrographs) and quality (pollutographs) values.

This manual presents utilization of those capabilities to analyze the combined sewers of the MDC area.

Complete documentation of the technical concepts and user instructions are presented in the following references published by the Water Quality Office, Environmental Protection Agency as part of their Water Pollution Control Research Series:

11024DOC07/71
Storm Water Management Model, Vol. I - Final Report; by Metcalf & Eddy Engineers, Palo Alto, California

11024DOC08/71
Storm Water Management Model, Vol. II - Verification and Testing; by Metcalf & Eddy, Inc., Palo Alto, California

11024DOC09/71
Storm Water Management Model, Vol. III - User's Manual; by Metcalf & Eddy Engineers, Inc., Palo Alto, California

11024DOC10/71
Storm Water Management Model, Vol. IV - Program Listing; by Metcalf & Eddy Engineers, Palo Alto, California

C. PURPOSE:

EPART is used as an analytic tool to determine those pipes in the sewer network that are inadequate to transport the design storm and to examine the conditions in a sewer system under alternative stormwater routing and pipe replacement strategies for remedial action.

D. CAPABILITIES AND FEATURES:

EPART is organized into separate control and computational blocks, each with certain capabilities:

1. The EXECUTIVE Block is the main control block with the following duties:

   a. Assigns logical file numbers (type/disk/drum)

   b. Maintains control of the execution of all other blocks

   c. Produces user-selected results as plots on the line printer

2. The RUNOFF Block computes the stormwater runoff from a given design storm for each subcatchment and stores the results in the form of hydrographs and pollutographs at inlets to the main sewer system to be used by TRANSPORT.

3. The TRANSPORT Block performs the flow and pollutants routing in the main sewer system picking up the runoff results as input and producing hydrographs, pollutographs and flooding conditions. These hydrographs and pollutographs are then printed in tabular form at user-selected points. At other user-selected points, hydrographs and pollutographs are stored on a file normally so that subroutine GRAPH in the EXECUTIVE Block can produce printed plots at these points. Additionally, the TRANSPORT Block produces a table of pipes and conduits that are surcharged during the storm. For the conduits in which maximum depth of surcharge is exceeded additional flow is stored on streets and pavements. The volume (cubic feet) of stormwater retained on streets and pavements upstream of any undersized pipe for any time step during the flow routing is listed by the program.

4. The COMBINE block combines hydrographs and pollutographs from several input files onto one tape in the format accepted by EPART. This block is used to pick up output hydrographs and pollutographs from tributary combined sewer areas for which EPART model runs have already been completed for use as input to downstream combined sewer areas. The COMBINE block overcomes the problem of modeling a combined sewer system in which there are more than 150 sewer elements that can be modeled by using EPART (maximum 150 elements including manholes).

E. RESTRICTION AND LIMITATIONS:

The maximum and minimum values for all input items as well as any interdependencies of data items are described in Section II - USER INSTRUCTIONS.

EPART has no inherent data checking capability and incorrect data may provide erratic results. It is, therefore, imperative that a careful check of the input be carried out.

II. USER INSTRUCTIONS

A. PROGRAM PROCESSING:

This section describes the steps performed in EPART.

Execution begins with the EXECUTIVE Block where title and description cards are read and the logical file numbers are assigned for scratch files and for input/output for each program block used.

The next block is the RUNOFF Block where the outflow hydrograph and pollutographs from each subcatchment are computed at user-specified points in the system (manholes) and stored on the output file specified in the EXECUTIVE Block.

The COMBINE Block can be used following the RUNOFF Block to pick up from files hydrographs and pollutographs from combined sewer areas tributary to the area now being modeled by EPART and which have been modeled by EPART prior to this run. This allows a large combined sewer area to be subdivided into smaller tributary areas to overcome the maximum limit of 150 elements in the TRANSPORT Block.

COMBINE first reads the main control file giving a two-card title, the number of time steps, number of pollutants, time step length, time of day, the number of and a list of nonconduit numbers to be found in the input stream. The program searches the input stream file-by-file, filling arrays in core with the required data. When all input files are complete, the output tape is created from these arrays. Any error in consistency terminates processing. All hydrographs and pollutographs picked up by the COMBINE Block are also stored on the output file specified.

The TRANSPORT Block reads the above information as input hydrographs and pollutographs from the file specified in the EXECUTIVE Block (normally the same file number as the RUNOFF Block output file), and routes these hydrographs and pollutographs through the specified conveyance system (sewers, manholes, pump stations, and other structures) producing hydrographs and pollutographs at selected locations in the system and information on surcharge and flooding as a result of pipe inadequacies, wherever such occurs.

Finally, any hydrograph(s) and pollutographs to be processed further are stored on the output file specified within the EXECUTIVE Block. For

example, subroutine GRAPH may be called after the TRANSPORT Block to produce printer plots of selected hydrograph(s) and pollutographs stored on the TRANSPORT Block output file.

The run is terminated when the EXECUTIVE Block reads the END PROGRAM card.

B. LOGISTICS:

EPART will execute in 520K bytes (without overlays) for run times in the vicinity of 45 seconds (on an IBM System 360/75). Using a simple overlay structure, the program may be run in a 360K region with a negligible change in execution time.

The file requirements for EPART are shown on Table EPART-1.

TABLE EPART-1. FILE REQUIREMENTS

Name	Description	RECFM(1)	LRECL(2)	BLKSIZE(3)
FT05F001	Card input	-	-	-
FT06F001	Main print file	FBA	133	1,330(4)
FT01F001(4)	Scratch file	VBS	800(4)	7,204(4)
FT02F001(4)	Scratch file	VBS	800(4)	7,204(4)
FT03F001(4)	Scratch file	VBS	800(4)	7,204(4)
FT04F001(4)	Scratch file	VBS	800(4)	7,204(4)
FT13F001(4)	Scratch file	VBS	800(4)	7,204(4)
FT08F001(4)	Input/Output file	VBS	800(4)	7,204(4)
FT09F001(4)	Input/Output file	VBS	800(4)	7,204(4)

1. Record format.
2. Logical record length.
3. Block size.
4. Suggested values - user option.

Every output manhole for which hydrographs and pollutographs are to be picked up by the COMBINE Block for use later must be found once and only once on the input file specified in the COMBINE Block.

The following limits are imposed based on the capacity of EPART by the COMBINE Block:

1. 150 time steps maximum

2. 60 manholes output maximum

3. 3 pollutants maximum

4. 16 input files are allowed

5. Input file number 5 is assumed to be a card file

6. All others are assumed to be EPART format tape or disk file

7. All inputs must agree with the number of pollutants, number of time steps, and time step length

Inconsistencies with any of the above rules except rule (5) will generate an explanatory error message. Breaking rule (5) will generate a Fortran abend.

Note: Manholes input from cards can be placed in file FT05 F001 behind the control input by using 5 as one of the input file numbers of on card 3 of the control cards.

C. INPUT DATA REQUIREMENTS:

The input data required for each block are described on Figures EPART-1 through 5 and Tables EPART-2 through 7. EPART has no inherent facility to perform data checking. Therefore, it is important that a check of input data be carried out.

D. INTERPRETATION OF RESULTS:

In addition to printing out the results of the simulation, input data are reproduced for verification of the data input. A check of this should be carried out prior to the analysis of results.

For the determination of sewer adequacy, two items of key information are provided; namely, outflow hydrographs at selected points and flooding conditions whenever such occur.

Flooding Conditions. An output table lists all surcharged elements for each time step and is produced according to the following logic: As

soon as the flow in any pipe or conduit exceeds the maximum capacity (QMAX) it is considered to be surcharged by the program. Pipe capacity under surcharged conditions are calculated using the slope of the hydraulic grade line based on surcharged depths in the downstream and upstream manholes. At each time step whether any pipe will be surcharged at the upstream end is determined by the depth of surcharge at the downstream manhole, pipe capacity and inflow at the upstream end. Thereafter, all flow in excess of pipe capacity under maximum surcharge depth in the upstream manhole is stored at the upstream manhole and the volume stored during each time-step is printed out. The surcharged element continues to flow full until the upstream depth of surcharge returns to zero at which point normal flow continues. In this table, the time shows the seconds from beginning of storm to the flooding condition. The surcharged element is the undersized pipe. The storage element is the manhole upstream of the surcharged pipe where the surcharge is stored. This surcharge is shown as the cumulative volume at each point in time.

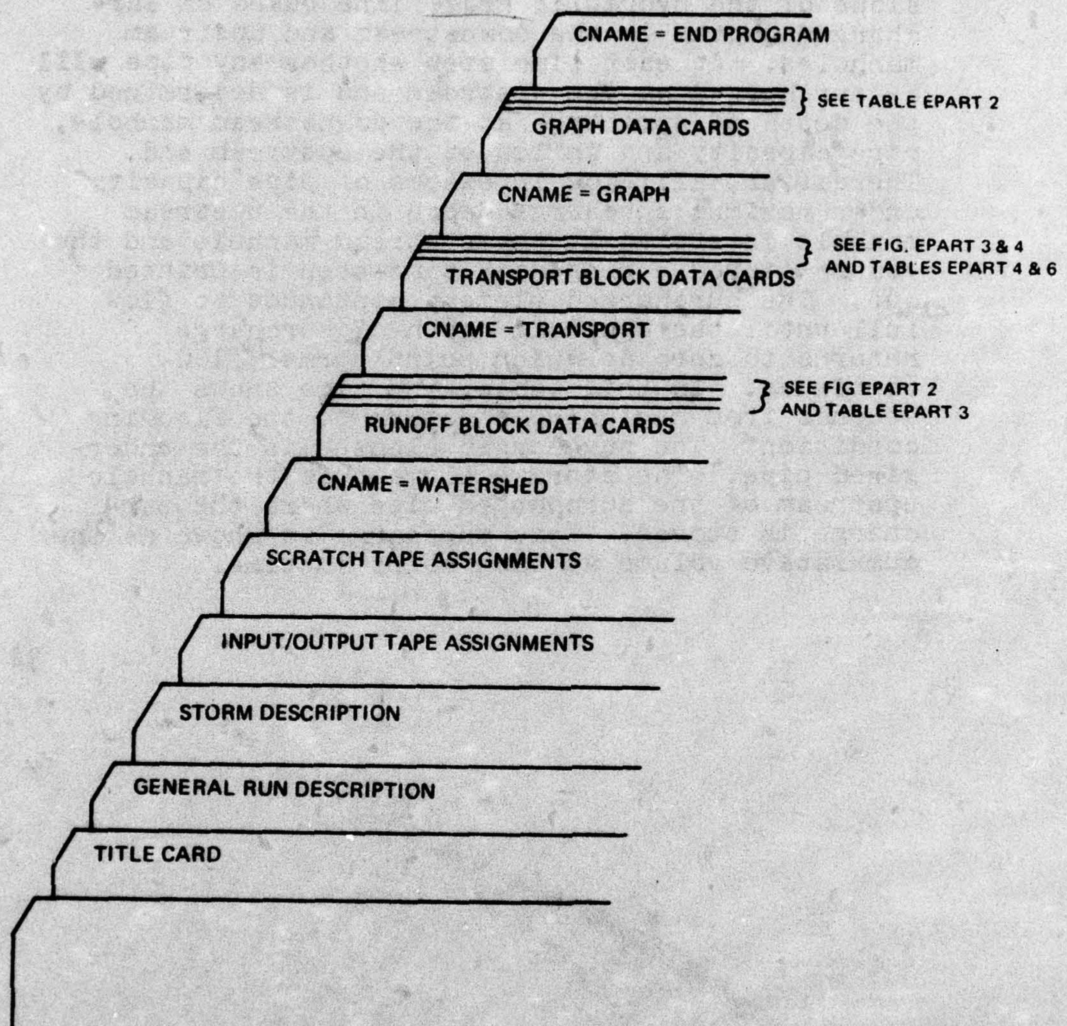

FIG. EPART 1 DATA DECK STRUCTURE FOR THE EXECUTIVE BLOCK

Table EPART-2. Executive Block Card Data

Card group	*Format*	*Card columns*	*Description*	*Units*	*Variable name*	*Maximum value*	*Default value*
1	10A4	1-40	Title Card, title of the area being studied.	–	TITLE1	–	none
2			General information about the studied area.				
	I5	1-5	Demonstration series number.	–	NSERYS	–	none
	8A4	6-40	Description of run.	–	DESCR	–	none
	I5	41-45	Number of storms being studied.	–	NSTRMS	10	none
			REPEAT FOR THE NUMBER OF STORMS.				
3			Storm data cards.				
	8A4	1-32	Description of storm.	–	STORM	–	none
4			I/O tape/disk assignments.				
	20I4	1-4	Input tape assignment for first block to be run.	–	JIN(1)	99	none
		5-8	Output tape assignment for first block to be run.	–	JOUT(1)	99	none
		9-12	Input tape assignment for second block to be run (usually the same as the output tape from first block).	–	JIN(2)	99	none

Table EPART-2 (Continued). Executive Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
		13-16	Output tape for second block to be run.	–	JOUT(2)	99	none
		⋮			⋮		
		77-80	Output tape for tenth block to be run.	–	JOUT(10)	99	none
5			Scratch tape/disk assignments.				
	20I4	1-4	First scratch tape assignment.	–	NSCRAT(1)	99	none
		5-8	Second scratch tape assignment.	–	NSCRAT(2)	99	none
		9-12	Third scratch tape assignment.	–	NSCRAT(3)	99	none
		13-16	Fourth scratch tape assignment.	–	NSCRAT(4)	99	none
		17-20	Fifth scratch tape assignment.	–	NSCRAT(5)	99	none
			REPEAT CARD 6 FOR EACH BLOCK TO BE CALLED!				
6			Control cards indicating which blocks in the program are to be called.				
	20A4	1-80	Name of block to be called.[1] =WATERSHED for Runoff Block, =COMBINE for Combine Block, =TRANSPORT for Transport Block, =GRAPH for GRAPH subroutines. =ENDPROGRAM for ending the storm water simulation.	–	CNAME	–	none

1. Name must start in Column 1. GRAPH may be called more than once.

Table EPART-2 (Continued). Executive Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
			INSERT THE REMAINING CARDS. IF CARD GROUP 6 INCLUDES CNAME = GRAPH, IMMEDIATELY FOLLOWING EACH GRAPH CARD.				
7			Control card.				
	4I5	1-5	Tape/disk (logical unit) assignment where graph information is stored.	–	NTAPE	99	none
		6-10	Number of curves of a graph.	–	NPCV	5	5
		11-15	Number of pollutants to be plotted	–	NQP	3	0
		16-20	Number of inlets to be plotted.	–	NPLOT	–	All curves on file.
			IF NPLOT = 0 (OR BLANK) DELETE THIS CARD.				
8			Inlet selection card.				
	16I5	1-5	First inlet number to be plotted.	–	IPLOT(1)	–	none
		6-10	Second inlet number to be plotted.	–	IPLOT(2)	–	none
		⋮			⋮		
			Last inlet number to be plotted.	–	IPLOT(NPLOT)	–	none
9			Title card.				
	18A4	1-72	Title printed with the plots.	–	TITL	–	none

Table EPART-2 (Continued). Executive Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
10			Horizontal axis label.				
	20A4	1-80	Horizontal axis label.	–	HRIZ	–	none
			Repeat NQP + 1 Times[1]				
11			Vertical axis label.				
	2A4	1-8	Line 1 of vertical axis label.	–	VERT(1)	–	none
		9-16	Line 2 of vertical axis label.	–	VERT(2)	–	none
	3A4	17-28	Line 3 of vertical axis label.	–	VERT(3)	–	none

1. The first plot to be printed is a flow hydrograph; the second is BOD; the third is SS; and the last is coliform.

NOTE: All non-decimal numbers must be right-justified.

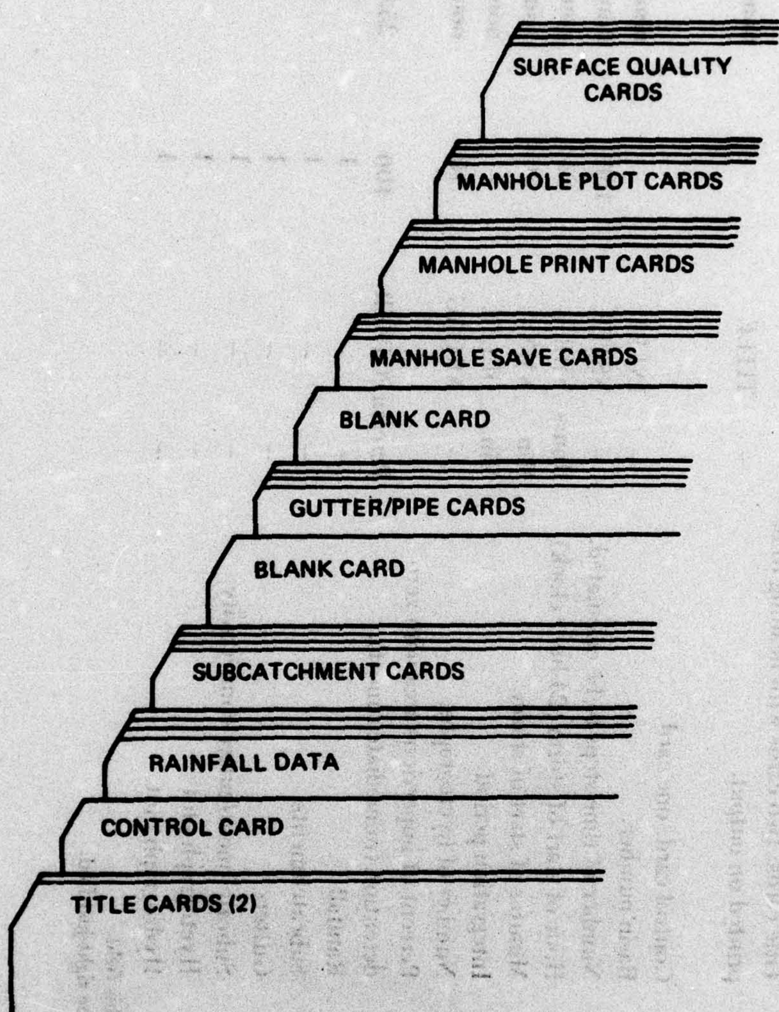

FIG. EPART 2 DATA DECK FOR THE RUNOFF BLOCK

Table EPART-3. Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
1	20A4		Title cards: two cards with heading to be printed on output.	–	TITLE	–	none
2			Control card: one card.				
	2I5	1-5	Basin number.	–	INLET	–	none
		6-10	Number of time-steps to be calculated.	–	NSTEP	150	none
	I3	11-13	Hour of start of storm (24-hour clock).	Hours	NHR	24	none
	I2	14-15	Minutes of start of storm.	Min.	NMN	59	none
	F5.1	16-20	Integration period.	Min.	DELT[1]	–	none
	I5	21-25	Number of hyetographs.	–	NRGAG	10	none
	F5.0	26-30	Percent of impervious area with zero detention (immediate runoff).	Percent	PCTZER	100	25.0
	I1	35	Rainfall	–	–	1	0
		40	Subcatchments	–	–	1	0
		45	Gutters	–	–	1	0
		50	Subcatchment/gutter connectivity	–	–	1	0
		55	Hyetograph plot	–	–	1	0
		70	Hydrograph plot	–	–	1	0

1. Decimal point should be punched in this field.

NOTE: All non-decimal numbers must be right-justified.

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
3			Rainfall control card.				
	I5	1-5	Number of data points for each hyetograph.	–	NHISTO	160	none
	F5.0	6-10	Time interval between values.	Min.	THISTO[1]	–	none
			REPEAT CARD GROUP 4 FOR EACH HYETOGRAPH.				
4			Rainfall hyetograph cards: 10 intervals per card.				
	10F5.0	1-5	Rainfall intensity, first interval	In./hr	RAIN(1)[1]	–	none
		6-10	Rainfall intensity, second interval	In./hr	RAIN(2)[1]	–	none
		11-15	Rainfall intensity, third interval	In./hr	RAIN(3)[1]		
		16-20	Rainfall intensity, fourth interval	In./hr	RAIN(4)[1]	–	none
		⋮			⋮		
			REPEAT CARD 5 FOR EACH SUBCATCHMENT.				
5			Subcatchment cards (3I5, 10F5.0, F10.5): one card per subcatchment.				
	3I5	1-5	Hyetograph number (Based on the order in which they are read in).	–	JK	10	1

1. Decimal point should be punched in this field.

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
		6-10[2]	Subcatchment number.	–	N	160	none
		11-15[2]	Gutter or manhole number for drainage.	–	NGOTO	160	none
	10F5.0	16-20	Width of subcatchment.	Ft	WWIDTH= W1[1]	–	none
		21-25	Area of subcatchment.	Acre	WAREA= W2[1]	–	none
		26-30	Percent imperviousness of subcatchment.	Percent	PCIMP=W3[1]	<100	none
		31-35	Ground slope.	Ft/ft	WSLOPE= W4[1]	–	0.030
		36-40	Impervious area } Resistance Factor.		W5 =W5[1]	–	0.013
		41-45	Pervious area } Resistance Factor.		W6 =W6[1]	–	0.250
		46-50	Impervious area } Retention storage.	In.	WSTORE= W7[1]	–	0.062
		51-55	Pervious area } Retention storage.	In.	WSTORE= W8[1]	–	0.250
		56-60	Maximum infiltration rate.	In./hr	WLMAX= W9[1]	–	3.00
		61-65	Minimum infiltration rate.	In./hr	WLMIN= W10[1]	–	0.52
	F10.5	66-75	Decay rate of infiltration.	1/sec	DECAY= W11[1]	–	0.00115

1. Decimal point should be punched in this field.
2. Need one inlet or gutter/pipe for each subcatchment basin.

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
6			Blank card to terminate subcatchment cards: one card.	–	–	–	–
			REPEAT CARD 7 FOR EACH GUTTER/PIPE				
7			Gutter/pipe cards: one card per gutter/pipe (if none, leave out).				
	4I5	1-5	Hyetograph number.	–	NHYET	10	none
		5-10	Gutter number.	–	N	160	none
		11-15	Gutter or manhole number for drainage.	–	NGOTO	160	none
		16-20	= 1 for gutter = 2 for pipe.	–	NP	–	none
	7F8.0	21-28	Bottom width of gutter or pipe diameter.	Ft	GWIDTH= G1[1]	–	none
		29-36	Length of gutter	Ft	GLEN =G2[1]	–	none
		37-44	Invert slope	Ft/ft	GSLOPE= G3[1]	–	none
		45-52	Left-hand side slope	Ft/ft	GS1 =G4[1]	–	none
		53-60	Right-hand side slope	Ft/ft	GS2 =G5[1]	–	none
		61-68	Manning's coefficient.		GN =G6[1]	–	none
		69-76	Depth of gutter when full.	In.	DFULL =G7[1]	–	10

1. Decimal point should be punched in this field.

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
8[1]			Blank card to terminate gutter cards: one card.	–	–	–	–
9			Manhole save control: one card.				
	I5	1-5	Number of inlet manholes for which entering flows are to be saved on peripheral storage for TRANSPORT.	–	NSAVE	–	none
			IF NSAVE=0, SKIP CARDS 10				
10			Manhole save cards: 16 values per card.				
	16I5	1-5	Inlet manhole numbers for which entering flows are saved (same elements that are used by TRANSPORT).	–	ISAVE(1)	–	none
		6-10		–	ISAVE(2)	–	none
		11-15		–	ISAVE(3)	–	none
		⋮			⋮		
				–	ISAVE (NSAVE)	–	none
11			Manhole print control: one card.				
	2I5	1-5	Number of inlet manholes for which entering flows are to be printed.	–	NPRNT	–	none
		6-10	Number of time-steps between printings.	–	INTERV	–	none

1. Need this card even though there are no gutter/pipe cards.

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
			IF NPRNT=0, SKIP CARDS 12				
12			Manhole print cards: 16 values per card.				
	16I5	1-5	Inlet manhole numbers for which entering flows are to be printed.	–	IPRNT(1)	–	none
		6-10		–	IPRNT(2)	–	none
		11-15		–	IPRNT(3)	–	none
		⋮			⋮		
					IPRNT (NPRNT)	–	none
13			Manhole plot control: one card.				
	3I5	1-5	Number of inlet manholes for which entering flows are to be plotted.	–	NPLOT	25	none
		6-10	Number of curves per figure.	–	NPCV	5	1
			IF NPLOT=0, SKIP CARDS 14.				
14			Manhole plot cards: 16 values per card.				
	16I5	1-5	Inlet manholes in which entering flows are to be plotted.	–	IPLOT(1)	–	none
		6-10		–	IPLOT(2)	–	none
		11-15		–	IPLOT(3)	–	none
		⋮			IPLOT (NPLOT)	–	none

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
			THE FOLLOWING CARDS ARE SURFACE QUALITY DATA.				
15			Control card. Use blank card if no surface quality desired.				
	2I5	1-5	Number of subareas (may exceed number of subcatchments due to multiple land uses)	–	KTNUM	160	none
		6-10[1]	Number of inlets.	–	NINLTS	50	none
	F5.0	11-15[1]	Time interval.	Min.	DT	–	none
	4I5	16-20[1]	Hour of start of storm (24-hr clock).	Hour	KHOUR	24	none
		21-25[1]	Minute of start of storm.	Min.	KMIN	59	none
		26-30[1]	Number of time-steps.	–	NTSTEP	150	none
		31-35	Use 1 for printing output in sentence form, 0 for printing in table form.	–	NPRINT	1	none
16			Cleaning data card. Omit if no surface quality dcsired.				
	2F10.0	1-10	Number of dry days prior to this storm in which the accumulative rainfall is <1.0 in.	–	DRYDAY	–	none
		11-20	Cleaning frequency.	Days	CLFREQ	–	none
	I5	21-25	Number of street sweeper passes.	–	NOPASS	–	none

1. These values must be the same as in card group 2.

Table EPART-3 (Continued). Runoff Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
17			Catchbasin data card. Omit if no surface quality desired.				
	3F10.0	1-10	Number per acre.	–	CBDEN	–	none
		11-20	Concentration of BOD of the stored water in each catchment basin.	Mg/L	CBBOD	–	none
		21-30	Stored volume in each catchment basin.	Gal.	CBVOL Cannot be o	–	none
			REPEAT DATA CARD 18 FOR EACH SUBAREA. (Maximum = 160 subareas).				
18			Subarea data card. Omit if no surface quality desired.				
	3I5	1-5	Number of this subarea.	–	KNUM	160	none
		6-10	Inlet number of this subarea.[1]	–	INPUT	–	none
		11-15	Land use = 1 for single family residential = 2 for multi-family residential = 3 for commercial = 4 for industrial = 5 for undeveloped or park lands.	–	KLAND	5	none
	2F10.2	16-25	Area of this subarea.	Acres	ASUB	–	none
		26-35	Total length of gutters for each subarea.	Hundreds of ft	GUTTER	–	none
			END OF RUNOFF BLOCK CARDS.				

1. All subareas with the same inlet number must be placed together and these groups must be in the order in which the inlets are saved as described by card group 10.

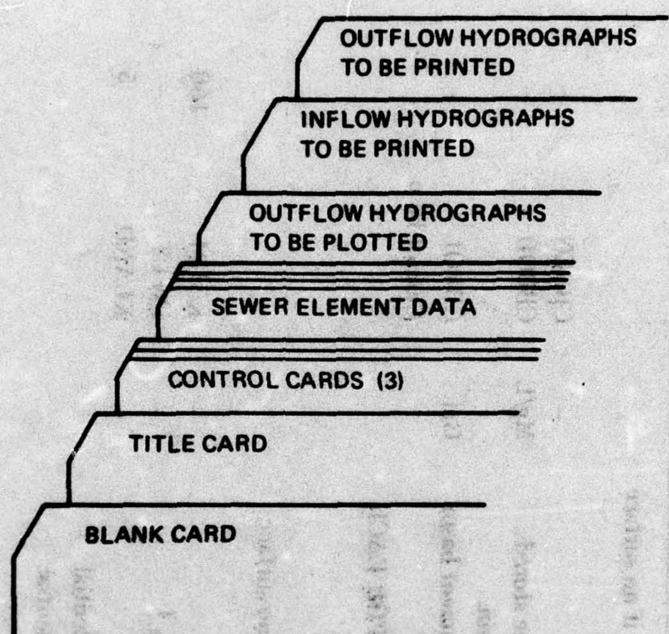

FIG. EPART 3 DATA DECK FOR THE TRANSPORT BLOCK

Table EPART-4. Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
1			Blank card.				
11	20A4		Title card containing a one-line heading to be printed above output. A numeral 1 should be placed in card column 1 for neat spacing print out.	–	TITLE	–	none
12			Execution control data.				
	16I5	3-5	Total number of sewer elements.	–	NE	150	none
		8-10	Total number of time-steps.	–	NDT	150	none
		14-15[1]	Total number of non-conduits into which there will be input hydrographs (minimum = 1).	–	NINPUT	60	none
		19-20	Total number of non-conduit elements at which input hydrographs are to be printed out (minimum = 1).	–	NNYN	10	none
		24-25	Total number of conduits & manholes elements at which routed hydrographs are to be printed out (minimum = 1).	–	NNPE	50	none
		30[2]	Total number of non-conduit elements at which flow is to be transferred to graph by tape (minimum = 1).		NOUTS	5	none

1. Must be the same as in the RUNOFF Block (NSAVE).
2. These are the only points that can be plotted by subroutine GRAPH after being routed by TRANSPORT.

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
		35	Control parameter for program-generated error messages concerning irregularities occurring in the execution of the flow routing scheme, i.e., NPRINT=0 to suppress messages, NPRINT=1 to print messages from ROUTE NPRINT=2 to print messages from ROUTE and TRANS.	–	NPRINT	2	0
		40	Total number of pollutants being routed (BOD, SS, coliform)	–	NPOLL	3	0
		45	Total number of iterations to be used in routing routine (4 recommended).	–	NITER	–	4
13			Execution control data.				
	8F10.5	1-10	Size of time-step for computations	sec.	DT	–	none
		11-20	Allowable error for convergence of iterative methods in routing routine (0.0001 recommended).	–	EPSIL	–	0.0001
		21-30	Total number of days (dry weather days) prior to simulation during which solids were not flushed from the sewers	–	DWDAYS	–	none
14			Execution control data.				
	16I5	5	Control parameter specifying means to be used in transferring inlet hydrographs, i.e.,	–	NCNTRL	1	0

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
			NCNTRL=1, normal transfer by tape or disk. NCNTRL=0, special transfer requiring additional input specifications.				
		10	Control parameter in estimating groundwater infiltration inflows, i.e., NINFIL=1, infiltration to be estimated (subroutine INFIL called), NINFIL=0, infiltration not estimated (INFIL not called and corresponding data omitted).	–	NINFIL	1	0
		15	Control parameter in estimating sanitary sewage inflows, i.e., NFILTH=1, sewage inflows to be estimated (subroutine FILTH called) NFILTH=0, sewage inflows not estimated (FILTH not called and corresponding data omitted).	–	NFILTH	1	0
		20	Control parameter concerning printed output, i.e., JPRINT=1, flows and concentration printed out in tabular form, JPRINT=0, flows and concentration not printed or plotted.	–	JPRINT	–	0

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
	I5	25	Inflow Pollutograph units (1=mg/L, 2=lb/min, 0=both)	-		2	0
	I5	30	Outflow Pollutograph units (1=mg/L, 2=lb/min, 0=both)	–		2	0
			REPEAT CARD GROUP 15 FOR EACH NUMBERED SEWER ELEMENT				
15			Sewer element data (max. 150 elements).				
	5I4	1-4	External element number. (Must be a positive numeral.)	–	NOE	1,000	none
			External number(s) of upstream element(s). Up to three are allowed. A zero denotes no upstream element.				
		5-8	First of three possible upstream elements.	–	NUE(1)	1,000	none
		9-12	Second of three possible upstream elements.	–	NUE(2)	1,000	none
		13-16	Third of three possible elements.	–	NUE(3)	1,000	none
		17-20	Classification of element type. Obtain value from Table EPART-5.	–	NTYPE	20	16

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
			THE FOLLOWING VARIABLES ARE DEFINED BELOW FOR CONDUITS ONLY. REFER TO TABLE EPART-6 FOR REQUIRED INPUT FOR NON-CONDUITS.				
	7F8.3	21-28	Element length for conduit.	Ft	DIST	–	none
		29-36	First characteristic dimension of conduit. See Figure EPART-4 and Table EPART-5 for definition.	Ft	GEOM1	–	0.0
		37-44	Invert slope of conduit (must be positive)	Ft/100 ft	SLOPE	–	0.1
		45-52	Manning's roughness of conduit.	–	ROUGH	–	0.013
		53-60	Second characteristic dimension of conduit. See Figure EPART-4 and Table EPART-5 for definition. (Not required for some conduit shapes.)	Ft	GEOM2	–	none
		61-68	Number of barrels for this element. The barrels are assumed to be identical in shape and flow characteristics.	–	BARREL	–	1.0
		69-76	Third characteristic dimension of conduit. See Figure EPART-4 and Table EPART-5 for definition. (Not required for some conduit shapes.)	Ft	GEOM3	–	none

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
	F4.0	77-80	Max. allowed water surface elevation at upstream manhole above top of conduit before ponding on street outside manhole	Ft	WSELMX	–	10.0
27			List of external non-conduit element numbers at which outflows are to be transferred to graph block.		JN		
	16I5	1-5	First element number.	–	JN(1)	–	none
		6-10	Second element number.	–	JN(2)	–	none
		⋮	Last element number.	–	JN(NOUTS)	–	none
28			List of external non-conduit element numbers at which input hydrographs are to be stored and printed out.		NYN		
	16I5	1-5	First input location number.	–	NYN(1)	–	none
		6-10	Second input location number.	–	NYN(2)	–	none
		⋮	Last input location number.	–	NYN(NNYN)	–	none
29			List of external conduit and manhole numbers at which output hydrographs and pollutographs are to be stored and printed out.		NPE		

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
	16I5	1-5	First output location number.	–	NPE(1)	–	none
		6-10	Second output location number.	–	NPE(2)	–	none
		.			.		
		.	Last output location number.	–	NPE(NNPE)	–	none
			IF SUBROUTINE INFIL IS TO BE CALLED (NINFIL = 1), INSERT CARDS 30 THROUGH 32, OTHERWISE OMIT.				
30			Estimated infiltration.				
	10F8.1	1-8	Base dry weather infiltration.	Gpm.	DINFIL	–	0.0
		9-16	Groundwater infiltration.	Gpm.	GINFIL	–	0.0
		17-24	Rainwater infiltration.	Gpm.	RINFIL	–	0.0
31			Control parameters.				
	I5	3-5	Day of estimate	–	NDYUD[1]	365	none
	6F8.1	6-13	Peak residual moisture.	Gpm.	RSMAX	–	0.0
		14-21	Average joint distance	Ft.	ULEN	–	6.0
32			Monthly degree-days.		NDD		
	16I5	1-5	July degree-days.		NDD(1)	–	none

1. Day one is July 15 and there are 365 days/year.

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
		6-10	August degree-days.		NDD(2)	–	none
		⋮			⋮		⋮
		56-60	June degree-days.		NDD(12)	–	none
			IF SUBROUTINE FILTH IS TO BE CALLED (NFILTH = 1), INSERT CARD GROUPS 33 TO 44, OTHERWISE OMIT.				
33			Factors to correct yearly average sewage flows to daily averages by accounting for daily variations throughout a typical week.				
	7F10.0	1-10	Flow correction for Sunday.	–	DVDWF(1)	–	0.0
		11-20	Flow correction for Monday.	–	DVDWF(2)	–	0.0
		⋮			⋮		
		61-70	Flow correction for Saturday.	–	DVDWF(7)	–	0.0
34			Factors to correct BOD yearly averages to daily averages.				
	7F10.0	1-10	BOD correction for Sunday.	–	DVBOD(1)	–	0.0
		⋮					⋮
		61-70	BOD correction for Saturday.	–	DVBOD(7)	–	0.0

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
35			Factors for correction of yearly SS averages to daily averages.				
	7F10.0	1-10	SS correction for Sunday.	–	DVSS(1)	–	0.0
		⋮					⋮
		61-70	SS correction for Saturday.	–	DVSS(7)	–	0.0
36			Factors to correct daily average sewage flow to hourly averages by accounting for hourly variations throughout a typical day (3 cards needed).				
	8F10.0	1-10	Midnight to 1 a.m. factor (first card).	–	HVDWF(1)	–	0.0
		⋮					⋮
		1-10	8 a.m. to 9 a.m. factor (second card).	–	HVDWF(9)	–	0.0
		⋮					⋮
		1-10	4 p.m. to 5 p.m. factor (third card).	–	HVDWF(17)	–	0.0
37			Factors for BOD hourly corrections (3 cards needed).				
	8F10.0	1-10	Midnight to 1 a.m. factor (first card).	–	HVBOD(1)	–	0.0
		⋮					⋮
		71-80	11 a.m. to midnight factor (third card).	–	HVBOD(24)	–	0.0

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
38			Factors for SS hourly corrections (3 cards needed).				
	8F10.0	1-10	Midnight to 1 a.m. factor (first card).	–	HVSS(1)	–	0.0
		⋮					⋮
		71-80	11 a.m. to midnight factor (third card).	–	HVSS(24)	–	0.0
			INCLUDE ONLY WHEN 3 POLLUTANTS ARE SPECIFIED.				
39			Factors for E. coli hourly corrections (3 cards needed).				
	8F10.0	1-10	Midnight to 1 a.m. factor (first card).	–	HVCOLI(1)	–	0.0
		71-80	11 a.m. to midnight factor (third card).	–	HVCOLI(24)	–	0.0
40			Study area data.				
	6I5	1-5	Total number of subareas within a given study area in which sewage flow and quality are to be estimated.	–	KTNUM	150	None
		6-10	Indicator as to whether study area data, such as treatment plant records, are to be used to estimate sewage quality, i.e., KASE = 1, yes, KASE = 2, no.	–	KASE	2	1

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
		11-15	Total number of process flows within the study area for which data are included in one of the following card groups.	–	NPF		0
		16-20	Number indicating the day of the week during which simulation begins (Sunday = 1).	–	KDAY	7	0
		21-25	Number indicating the hour of the day during which simulation begins (1 a.m. = 1).	–	KHOUR	23	0
		26-30	Number indicating the minute of the hour during which simulation begins.	–	KMINS	59	0
	2F5.1	31-35	Consumer Price Index.	–	CPI	–	109.5
		36-40	Composite Construction Cost Index.	–	CCCI	–	103.0
	F10.3	41-50	Total population in all areas (thousands).	–	POPULA	–	none
			IF KASE = 1, INCLUDE CARD GROUPS 41, 42 AND 43.				
41			Average study area data.				
	3F10.0	1-10[1]	Total study area average sewage flow, i.e., from treatment plant records.	Cfs	ADWF		0.0
		11-20	Total study area average BOD.	Mg/L	ABOD	–	none
		21-30	Total study area average SS.	Mg/L	ASUSO	–	none
	E10.2	31-40	Total coliforms	MPN/100	ACOCI	–	none

1. If ADWF = 0.0, then total BOD, SS, and COLI will = 0.0.

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
42			Categorized study area data.				
	8F8.0	1-8	Total study area from which ABOD and ASUSO were taken.	Acres	TOTA	–	none
		9-16	Total contributing industrial area.	Acres	TINA	–	none
		17-24	Total contributing commercial area.	Acres	TCA	–	none
		25-32	Total contributing high income (above $15,000) residential area.	Acres	TRHA	–	none
		33-40	Total contributing average income (above $7,000 but below $15,000) residential area.	Acres	TRAA	–	none
		41-48	Total contributing low income (below $7,000) residential area.	Acres	TRLA	–	none
		49-56	Total area from the above three residential areas that contribute additional waste from garbage grinders.	Acres	TRGGA	–	none
		57-64	Total park and open area within the study area.	Acres	TPOA	–	none
			IF PROCESS FLOW DATA ARE AVAILABLE (NPF NOT EQUAL 0 AND KASE = 1), REPEAT CARD GROUP 43 FOR EACH PROCESS FLOW.				
43			Process flow characteristics.				

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
	I5	1-5	External manhole number into which flow is assumed to enter (maximum value = 150, minimum value = 1).	–	INPUT	150	none
	6F10.3	6-15	Average daily process flow entering the study area system.	Cfs	QPF	–	none
		16-25	Average daily BOD of process flow.	Mg/L	BODPF	–	none
		26-35	Average daily SS or process flow.	Mg/L	SUSPF	–	none
			REPEAT CARD GROUP 44 FOR EACH OF THE KTNUM SUBAREAS.				
44			Subarea data.				
	2I3	1-3	Subarea number.	–	KNUM	150	none
		4-6	External number of the manhole into which flow is assumed to enter for subarea KNUM (maximum value = 150, minimum value = 1).	–	INPUT	150	none
	3I1	7	Predominant land use within subarea.	–	KLAND	5	none
		8	Parameter indicating whether or not water usage within subarea KNUM is metered. METHOD = 1, metered water use. METHOD = 2, incomplete or no metering.	–	METHOD	2	2
		9	Parameter indicating units in which water usage estimates (WATER) are tabulated.		KUNIT	1	0

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
			KUNIT = 0, thousand gal./mo.				
			KUNIT = 1, thousand cf/mo.				
	13F5.1	10-14	Measured winter water use for subarea KNUM in the units specified by KUNIT (not required).		WATER	–	none
		15-19	Cost of the last thousand gal. of water per billing period for an average consumer within subarea KNUM (cents/1,000 gal.) (not required).		PRICE	–	none
		20-24	Measured average sewage flow from the entire subarea KNUM (not required).	Cfs	SEWAGE		none
		25-29	Total area within subarea KNUM (maximum = 200).	Acres	ASUB	–	none
		30-34[1]	Population density within subarea KNUM	Pop./acre	POPDEN	–	none
		35-39[1]	Total number of dwelling units within subarea KNUM.	–	DWLNGS	–	10.0/ac.
		40-44[1]	Number of people living in average dwelling unit within subarea KNUM.	–	FAMILY	–	3.0
		45-49[1]	Market value of average dwelling unit within subarea KNUM (thousands of dollars).		VALUE	–	20.0

1. Not required if KLAND greater than 2.

Table EPART-4 (Continued). Transport Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
		50-54[1]	Percentage of dwelling units possessing garbage grinders within subarea KNUM.		PCGG	–	none
		55-59[2]	Total industrial process flow originating within subarea KNUM.	Cfs	SAQPF	–	0.0
		60-64	BOD contributed from industrial process flow originating with subarea KNUM.	Mg/L	SADPF	–	none
		65-69	SS contributed from industrial process flow originating within subarea KNUM.	Mg/L	SASPF	–	none
		70-74	Income of average family living within.	–	XINCOM	–	VALUE/2.5
	I2	75-76	MSUBT = 0, subtotals not made. MSUBT = 1, subtotal made.	–	MSUBT	1	0
			END OF FILTH DATA CARDS.				

FOR GRAPHING TRANSPORT OUTPUT CALL GRAPH SUBROUTINE THROUGH THE EXECUTIVE BLOCK AS EXPLAINED IN TABLE EPART-2

1. Not required if KLAND greater than 2.
2. If SAQPF = 0.0, then DWBOD and DWSS will be zero for Land Use 4 (i.e., for industrial flows to be considered KLAND must equal 4).

Table EPART 5. Summary of Area Relationships and Required Conduit Dimensions*

Ntype	Shape	Area	Required dimensions, ft
1	Circular	$(\pi/4)(G1)^{(2)}$	GEOM1=Diameter
2	Rectangular	G1 (G2)	GEOM1=Height GEOM2=Width
3	Egg-shaped	$0.5105(G1)^{(2)}$	GEOM1=Height
4	Horseshoe	$0.829\ (G1)^{(2)}$	GEOM1=Height
5	Gothic	$0.655\ (G1)^{(2)}$	GEOM1=Height
6	Catenary	$0.703\ (G1)^{(2)}$	GEOM1=Height
7	Semielliptic	$0.785\ (G1)^{(2)}$	GEOM1=Height
8	Basket-handle	$0.786\ (G1)^{(2)}$	GEOM1=Height
9	Semi-circular	$1.27\ (G1)^{(2)}$	GEOM1=Height
10	Modified basket-handle	$G2(G1 + (\pi/8)G2)$	GEOM1=Side height GEOM2=Width
11	Rectangular, triangular bottom	$G2(G1 - G3/2)$	GEOM1=Height GEOM2=Width GEOM3=Invert height
12	Rectangular, round bottom	$\theta = 2\ \mathrm{ARSIN}\ (G2/2G3)$ $\mathrm{Area} = G1\ (G2) + (G3)^2 / 2(\theta - \mathrm{SIN}(\theta))$	GEOM1=Side height GEOM2=Width GEOM3=Invert radius
16	Manhole	–	**
18	Flow divider	–	**
20	Flow divider	–	**
21	Flow divider	–	**

*Refer to Figure EPART-4 for definition of dimensions, G1, G2, and G3.
**Refer to Table EPART-6 for definition of non-conduit data.

Table EPART-6. Parameters Required for Non-Conduits

NTYPE	Description	DIST	GEOM1	SLOPE	ROUGH	GEOM2	BARREL	GEOM3
16	Manhole	N.R.[1]	N.R.	N.R.	N.R.	N.R.	N.R.	N.R.
18	Flow divider	N.R.	Maximum undiverted flow. Inflow in excess of this value is diverted (cfs).	N.R.	N.R.	N.R.	N.R.	Number of element into which flows the undiverted flow (include decimal point).
20	Flow divider	Maximum inflow without flow over the weir (cfs).	Weir height, above zero flow depth (ft).	Maximum inflow through whole structure (cfs).	Weir constant times weir length (ft).	Depth in structure at time of maximum inflow (ft).	N.R.	Number of element into which flows the undiverted flow (weir flow is the diverted flow).

Table EPART-6 (Continued). Parameters Required for Non-Conduits

NTYPE	Description	DIST	GEOM1	SLOPE	ROUGH	GEOM2	BARREL	GEOM3
21	Flow divider	N.R.[1]	Maximum undiverted flow beyond which flow is split	N.R.	N.R.	Fraction of flow to GEOM3 when flow exceeds GEOM1	N.R.	Number of element into which flows the undiverted flow

1. N.R. = Not required.

NOTE: All elements require an element number (NOE) and type (NTYPE). Parameters for conduits are defined in Table EPART-5.

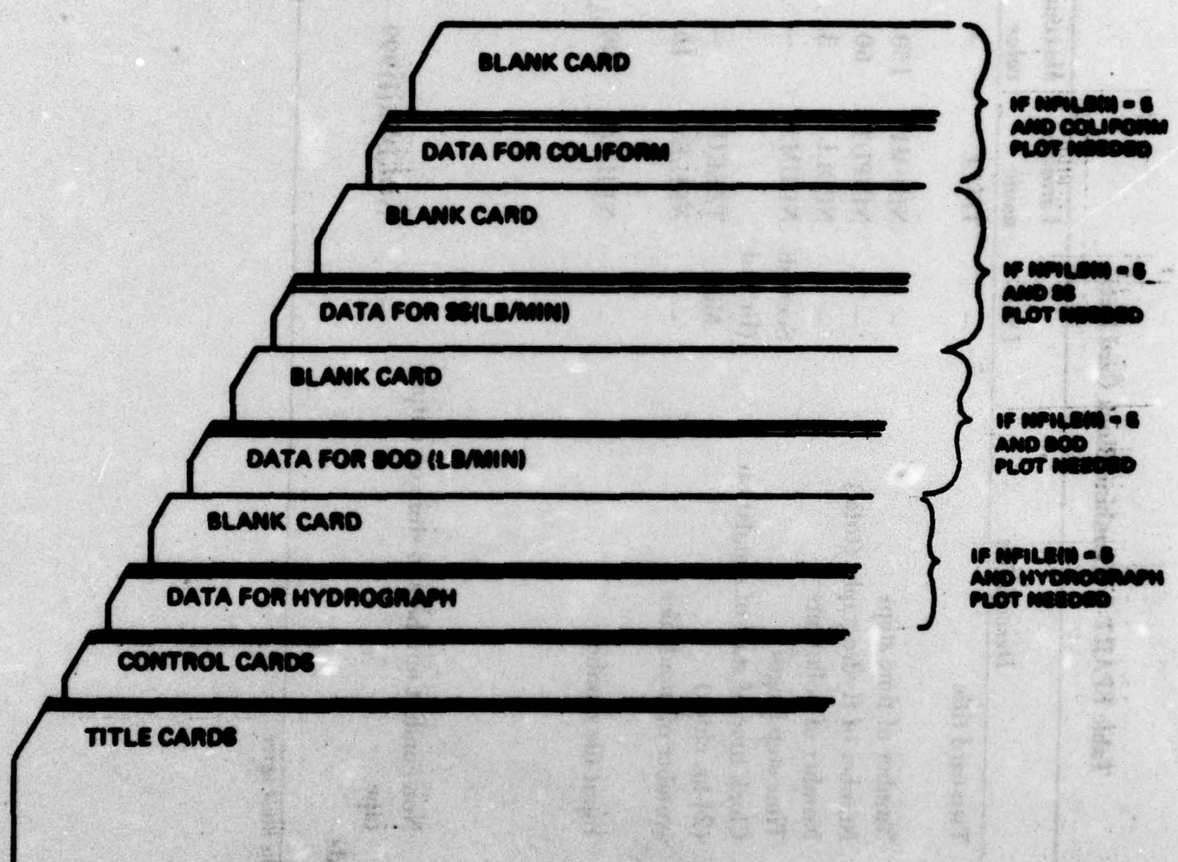

FIG. E PART-4 DATA DECK STRUCTURE FOR THE COMBINE BLOCK

Table EPART-1. Combine Block Card Data.

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
1	2(20A4)	1-80	Two-card title	–	TITLE	–	–
2	I10	1-10	Number of time steps	–	NDUM1[1]	150	–
	I10	11-20	Number of Hydrographs (total)	–	NINPUT	60	–
	I10	21-30	Number of pollutants	–	NPOLL[1]	3	–
	I10	31-40	Time step length	Seconds	NDUM2[1]	–	–
	F10.0	41-50	Clock time of start of simulation (24-hr. clock)	(Hr and Min)	TZERO	–	–
	I10	51-60	Number of input files	–	NFILES	16	–
3	16I5	1-5, 6-10,...	Input file number	–	NFILE(I)	99	Files 6 and 77 dedicated. NFILE (I)=5 is data cards
4	16I5	1-5, 6-10,... (as many cards as needed)	Non-conduit numbers to store on output tape	–	NORDER(I)	999	–

1. All input hydrographs and pollutographs must agree.

Table EPART-7 (Continued). Combine Block Card Data

Card group	Format	Card columns	Description	Units	Variable name	Maximum value	Default value
5	I5	1-5	Non-conduit (MH) number	–	MH	999	–
	I5	6-10	Number of time steps	–	NTIM	150	–
	I5	11-15	Length of time step	Seconds	LTIM	–	–
	I5	16-20	Number of pollutants	–	NP	3	–
6	10F8.0	1-8, 9-16,...	Data Card-10 items per card- This format used for: A. Hydrographs CFS RNOFF B. BOD Lb/Min PLUTO(*,1,*) C. SS Lb/Min PLUTO(*,2,*) D. Coliform MPN/Min PLUTO(*,3,*) Use as many cards as needed for each hydrograph and pollutograph at ten items per card. Follow each hydrograph and pollutograph with a blank card.				
7	80X	1-80	Blank Card (May be used for comments)				

APPENDIX F

INDEX OF MODELING PACKAGES

APPENDIX F

INDEX OF MODELING PACKAGES

The combined sewer systems in Metropolitan Boston were modeled using the EPA Storm Water Management Model to quantify the flows and pollutants discharged into the receiving waters during a 1-year 6-hour storm. The entire combined sewer area was subdivided into smaller areas as shown on Figure 2-1, to conform with the internal operational constraints of the model. These constraints limit the input data pertaining to representation of the physical system to 160 sewer elements, including manholes. Each of the subareas having combined sewers were modeled as a separate computer run and were given a designation for easy identification of computer outputs. Table F-1 lists the designations used for each of the 28 subareas together with the municipality in which they are located and the area description. The input data decks, their listings, and the results of computer modeling runs are not included in this report, but are on file with the Metropolitan District Commission.

The combined sewer system in Boston is complicated in that there are a number of major sewers originating in one part of the City and traveling across the City, with many interconnections, before overflowing to the receiving waters. Thus, even though the City was subdivided into smaller areas to conform to the limitation of a maximum of 160 sewer elements per model run, hydrographs and pollutographs from one subarea had to be routed through other subareas to reflect the actual operation of the sewer system. All sewers crossing from one subarea to another had hydrographs and pollutographs transferred via a manhole common to both areas as shown on Figure F-1. In Figure F-1 the numbers identify each transfer from one subarea to another and the arrows indicate the direction of the transfer.

It was necessary to model the subareas in the sequence shown in Table F-2 in order to generate any necessary hydrographs and pollutographs for transfer to adjacent areas:

TABLE F-1. LIST OF COMPUTER MODELING PACKAGES

No.	Computer modeling package designation	Municipality	Area description
1	B1	Boston	Charlestown area
2	B2	Boston	East Boston and Orient Heights area
3	B3	Boston	Brighton area
4	B4	Boston	Area tributary to Boston Marginal Conduit and Lowell Street, Boston
5	B5	Boston	Downtown Boston
6	B8	Boston	Parts of Roxbury and Dorchester
7	B8A	Boston	Parts of Roxbury and Jamaica Plain
8	B8B	Boston	Parts of Roxbury
9	B9	Boston	Parts of South Boston and Dorchester
10	B9A	Boston	Parts of South Boston (Reserved channel and Fort Point channel area)
11	B9B	Boston	Parts of South Boston (Beach areas)
12	B10	Boston	Parts of West Roxbury
13	B11	Boston	Most of Jamaica Plain
14	B12	Boston	Parts of Dorchester
15	B15	Boston	Parts of Roslindale and Hyde Park
16	B16	Boston	Parts of Dorchester and Mattapan

TABLE F-1 (Continued). LIST OF COMPUTER MODELING PACKAGES

No.	Computer modeling package designation	Municipality	Area description
17	BR	Brookline	Area tributary to St. Mary's Street combined sewer
18	C3	Cambridge	Area enclosed by Mass. Avenue, Cambridge Street, and Fifth Street
19	C7	Cambridge	Part of area tributary to BU facility
20	C8	Cambridge	Part of area tributary to BU facility
21	C9	Cambridge	Part of area tributary to BU facility
22	C10	Cambridge	Part of area tributary to BU facility
23	C12	Cambridge	Part of area tributary to BU facility
24	C14	Cambridge	Area tributary to Lowell Street
25	C16	Cambridge	Area tributary to Alewife Brook Conduit
26	CH	Chelsea	Downtown combined sewer area
27	S1 (1)	Somerville	Area tributary to McGrath Highway combined sewer and the Cambridge Marginal Conduit
28	S2	Somerville	Area tributary to Somerville Pretreatment Facility and Marginal Conduit

1. Includes Cambridge subarea C1.

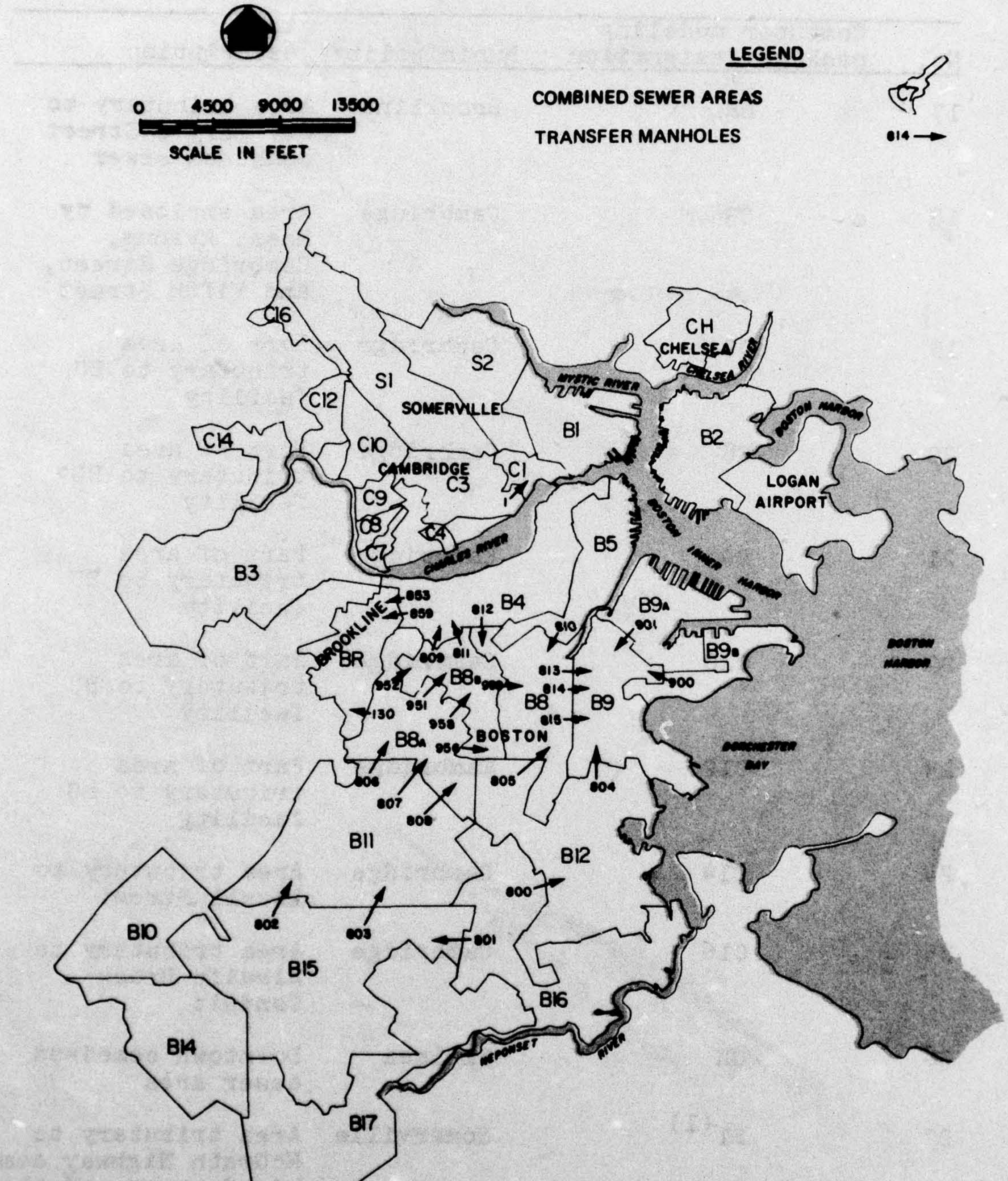

FIG. F-1 TRANSFER MANHOLES IN COMBINED SEWER MODELING

TABLE F-2. INTERRELATED SUBAREA MODELING SEQUENCE

Modeling order	Subarea designation
1	B5
2	B9A
3	B9B
4	B15
5	B16
6	B11
7	B12
8	B8A
9	B8B
10	B4
11	B8
12	B9
13	BR